LA--10576-MS

DE86 008575

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SUBSIDENCE AND SETTLEMENT
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by

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ABSTRACT

Subsidence and settlement are phenomena that are much more destructive than generally thought. In shallow land burials they may lead to cracking of the overburden and eventual exposure and escape of waste material. The primary causes are consolidation and cave-ins. Laboratory studies performed at Los Alamos permit us to predict settlement caused by consolidation or natural compaction of the crushed tuff overburden. We have also investigated the shear failure characteristics of crushed tuff that may lead to subsidence. Examples of expected settlement and subsidence are calculated based on the known geotechnical characteristics of crushed tuff. The same thing is done for bentonite/tuff mixes because some field experiments were performed using this additive (bentonite) to reduce the hydraulic conductivity of the crushed tuff. Remedial actions, i.e., means to limit the amount of settlement, are discussed. We finally discuss our field experiment, which studies the influence of subsidence on layered systems in general and on biobarriers in particular. The share of the produced cavities is compared with cavities produced by idealized voids in an idealized environment. Study of root penetration at subsidence sites gives us an indication of the remaining degree of integrity.

I. INTRODUCTION

A. Evidence of Occurrence

1. General. Engineering structures may cause foundation failures in one of two ways: excessive settlement or shear failure of the supporting soil. These failures are caused by (1) unexpected increase or decrease in soil moisture content, (2) compaction under unforeseen pressures, (3) soil heave caused by frost action and settlement caused by thaw, and (4) creep and slides resulting from shear failure (Jumikis 1968). The engineering structures involved may include whole cities (Mexico City), building complexes, or parts of buildings. Partial building subsidence is primarily caused by uneven soil settlement. The best known example of this is the Leaning Tower of Pisa, which is famous only because it has not fallen down in eight centuries despite its still settling foundations. Total subsidence in Mexico City has passed the 10-m mark since the beginning of this century, but uneven settlement or subsidence is much more damaging than total subsidence and resulted in disastrous ruptures of the sewer system and pipelines, causing cracking, tilting, and subsidence of monuments and buildings, old and modern. Such soil settlement in Mexico City can be studied as large-scale consolidation tests.

Vibrations can also have a densification effect on soils and can lead to subsequent settlement. The effects can be severe when the vibration frequency matches the soil's natural frequency. Soils often fail and settle disastrously as a result of earthquakes. Devastating landslides are often one of the results of such occurrences. Most earthquake accelerations, however, are too small to cause densification, and only large earthquakes will cause subsidence of the upper soil layer, which may amount to more than 1 m at large accelerations as occurred in Valdivia, Chile, in 1960 (Lambe and Whitman 1979). Natural or anthropogenic modification of the landscape, such as slopes or modification of supporting medium in landfills, may also be subject to failure. Differential settlements are usually structurally the most critical.

Of the three phases possibly present in a soil, only the solid phase controls the resistance to compression and shear. Water, present in a moist soil is highly incompressible, but as a liquid, it is, by definition, not

capable of resisting shear loads. Air, present in unsaturated soils, will not support compression or shear loads.

In a saturated soil, compression will be primarily caused by expulsion of water out of the soil voids. Under the influence of an externally applied load, the expulsion of water from the voids is highly dependent on the permeability of the medium. The extremely low permeability in the case of clay leads to a slow void contraction. The compression of saturated, low-permeability layers under a static pressure is known as consolidation. The consolidation rate depends on the compressibility of the soil (rate of decrease in volume with stress) and soil permeability, which, in turn, is dependent on the viscosity of the liquid (viscosity of water at 35⁰ C is half that at 5⁰ C). An increase in temperature increases the consolidation rate but does not affect total amount of consolidation (Head 1982).

The oedometer test maintains a constant stress until settlement is virtually complete and no evidence of neutral stress or pore pressure remains (Fig. 1). Initially, the stress is converted into increased pore pressure. As water is expelled out of the soil voids, the pore pressure gradually drops to zero. The results are read as a plot of void ratio vs time for a given total stress (pore stress + effective stress).

Failure to drain the pores will result in low shear resistance. The ability to resist shear loads is solely dependent on the mechanical interaction of the solid particles in the soil matrix. The presence of excess water reduces the effective stress responsible for the friction between solids.

Quantitative studies involving the physical and mechanical properties of soils and having direct application on the design or the construction of waste disposal facilities include hydraulic conductivity, consolidation, and shear strength. Long-term soil consolidation and shear failure will result in subsidence.

Several reports dealing with the hydraulic conductivity of crushed and solid Bandelier tuff, as well as that of adjacent soils, have been published (Abeelee 1979, Abeelee et al. 1981, Abrahams 1963, Abrahams et al. 1961,

Fig. 1. Hydromechanical analogy for consolidation.

Purtymun and Koopman 1965). Consolidation and shear strength are discussed below.

2. Shallow Land Burial. Uneven settlement or differential settlement is far more damaging to a pit overburden than is total settlement. Uneven settlement will lead to cracking of the overburden and eventual exposure and escape of waste material. Differential or partial settlement is very often described as subsidence, although we found the terms are often used interchangeably. The nonhomogeneity of the buried waste and containers is the major cause for differential settlement. This nonhomogeneity is also the cause of temporary arching and sudden collapse or subsidence. It is also the reason that differential settlement is so much more difficult to estimate.

Exposure of waste materials is studied in our field experiment. This study concerns the integrity of a biobarrier when collapse, subsidence, or disruption of a soil layer (e.g., a biobarrier) occurs.

B. Causes

The magnitude of soil settlements depends on the compressibility of the soil, moisture and temperature fluctuations in the soil, and the stresses applied upon it. Several broad causes for soil settlement are recognized:

1. Consolidation;
2. Lateral and upward expulsion of cohesionless or saturated soil masses;
3. Cave-ins resulting from
 - (a) Unbraced excavations, such as shallow land burial pits either before or after backfilling, caused by exceeding the shear strength of the slope;
 - (b) Rotting or degradation of the waste products serving as braces and support for the overburden or backfill;
 - (c) Slumping of the overburden, which is caused by movement of soil particles into existing interstices between waste containers;

- (d) Decreased soil shear strength through wetting;
- (e) Large-scale dewatering;

4. Inadequate soil compaction.

Table I, borrowed from Sowers (1979), indicates the principal causes of settlement; Table II indicates the pressures at which a typical material will fail.

C. Mechanisms

Settlement means some form of densification. Minimum density is obtained by measuring oven-dried soil, which has been poured into a container of known volume. Maximum density is obtained by vibrating that container according to (not entirely standardized) specification. In general, the smaller the particle size distribution, the lower the density. The relative density, D_r , of a granular medium or soil can be defined as a function of the void ratio, e , or

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100\%. \quad (1)$$

The density of granular soils (Table III) is characterized according to Lambe and Whitman (1979). Several interdependent mechanisms contribute to the densification of granular soils:

1. compression of air and water in the voids,
2. squeezing of air and water out of voids,
3. permanent deformation caused by crushing of particles,
4. elastic deformation caused by bending of particles, and
5. rearrangement of particles caused by sliding and rolling of particles relative to one another.

TABLE I
CAUSES OF SETTLEMENT

Cause	Form of Mechanism	Amount of Settlement	Rate of Settlement
Structural load	Distortion (change in shape of soil mass)	Compute by elastic theory (partly included in consolidation)	Instantaneous
	Consolidation: Initial	Stress-void ratio curve, time curve	Rapid
	Change in void ratio under stress Primary	Stress-void ratio curve	Compute from Terzaghi theory
	Secondary	Compute from log time-settlement	Compute from log time-settlement
Environmental load	Shrinkage (due to drying)	Estimate from stress-void ratio or moisture-void ratio and moisture loss limit-shrinkage limit	Equal to rate of drying. Seldom can be estimated
Load independent (but may be aggravated by load; often environment related, but not dependent	Consolidation (due to water table lowering) Reorientation of grains—shock and vibration	Compute from stress-void ratio and stress change Estimate limit from relative density (up to 60-70%)	Compute from Terzaghi theory Erratic, depends on shock, relative density
	Structural collapse—loss of bonding (saturation thawing, etc.)	Estimate susceptibility and possibly limiting amount	Begins with environmental change, rate erratic
	Raveling—erosion into openings, cavities	Estimate susceptibility but not amount	Erratic, gradual or catastrophic, often increasing
	Biochemical decay	Estimate susceptibility, possible limits	Erratic, often decreases with time
	Chemical attack	Estimate susceptibility	Erratic
	Mass collapse—collapse of sewer, mine, cave	Estimate susceptibility	Likely to be catastrophic
	Mass distortion—shear-creep or landslide in slope	Compute susceptibility from stability analysis	Erratic, catastrophic to slow
	Expansion—frost, clay expansion, chemical attack (resembles settlement)	Estimate susceptibility sometimes limiting amount	Erratic, increases with wet weather

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The property that influences deformation and consequent settlement to the greatest extent is the modulus of elasticity or the stress/strain modulus, E . The bearing capacity of a granular soil depends to a high degree on the internal friction angle or angle of repose (see Sec. II.B.) and on the relative density of the granular soil in question.

TABLE II

**FAILURE PRESSURES FOR
TYPICAL MATERIALS**

<u>Material</u>	<u>Pressure (kPa)</u>
Soft Clay	45
Submerged Loose Sand	60
Dry Loose Sand	100
Stiff Clay	175
Submerged Dense Sand	240
Hard Clay	400
Dry Dense Sand	500
Weathered Rock	500
Hard Rock	10 000

TABLE III

DENSITY OF GRANULAR SOILS

<u>D_r (%)</u>	<u>Description</u>
0-15	Very Loose
15-35	Loose
35-65	Medium
65-85	Dense
85-100	Very Dense

II. STUDIES AT LOS ALAMOS

A. Consolidation

Loading a soil with a manmade fill will cause deformation. The resulting total vertical surface deformation or displacement is described as settlement. Also, a lowering of the water table will cause an increase in the effective stress, σ_{eff} , and will cause settlements. The total settlement, ΔH , has three components (Holtz and Kovacs 1981):

ΔH_1 = immediate settlement or distortion,

ΔH_2 = time-dependent settlement or consolidation, and

ΔH_3 = secondary time-dependent settlement.

The distortion component can be estimated using the elastic theory where, according to Hooke's law,

$$\Delta H = \frac{PH}{EA}, \quad (2)$$

with

E = elasticity modulus in Pa,

P = load in N,

H = thickness of soil layer in m, and

A = area of soil under stress.

The elasticity modulus is determined by the slope of the initial stress/strain curves. This can be done by taking the initial slope of a stress/strain curve, called the tangent modulus or, because the stress/strain curves are not entirely linear, by taking the slope of the straight line from the origin to a certain stress, which is called the secant modulus (Holtz and Kovacs 1981).

When additional stress is applied to the saturated soil, the solid structure will not immediately support it because water will prevent compression. Pore pressure supports the applied load. As the water is forced out, the soil compresses and the solid structure assumes more and more of the load until the neutral stress becomes zero and the solid particles support the total load or effective stress. The neutral stress can be read by a piezometer. Because pore-water pressure measurements are not made in the oedometer, the degree of consolidation, U , is calculated directly from the change in height, H , of the sample, with $U = 0\%$ at the start of consolidation and $U = 100\%$ at its completion. The change in void ratio is

$$\Delta e = (1+e_0)\Delta H/H_0. \quad (3)$$

The time required to reach any percentage of consolidation for any thickness of a particular soil layer can be evaluated from the consolidation curve obtained in the laboratory. The time for any degree of consolidation will be a function of the square of the thickness of a particular soil layer and its permeability at that particular consolidation pressure, so that rate and amount of settlement of a structure can be calculated. This would enable

one to estimate whether settlements will be substantially completed during construction or how long the settlements will last after completion. Means for accelerating the consolidation, such as sand drains or wicks, may be considered.

After equilibrium is reached and the transfer from neutral to effective stress is complete, the test proceeds by addition of a new load increment and by allowing settlement to occur until equilibrium is reached under the new total stress, indicating the new consolidation is complete. For adequate computations of the coefficient of consolidation, C_v , standard load increments of $\Delta\sigma/\sigma = 1$ must be used. This value, C_v , varies for each stress increment and is, therefore, calculated every time a load increment is applied. A total final stress of 1 MPa was applied. The time rate for each settlement measurement during each load increment test was set at $\Delta t/t = 1$. It is important to remember that the rate of settlement is primarily a function of the compressibility and permeability of the medium. The coefficient of volume compressibility is $m_v = d\epsilon/d\sigma$ with $\epsilon = \Delta H/H$ the relative strain or compressibility. It is noteworthy that m_v is the reciprocal of the modulus of elasticity, compression, or constraint. If the void ratio at equilibrium is plotted against applied stress, the slope of the curve is termed the coefficient of compressibility:

$$a_v = de/d\sigma = m_v(1+e), \quad (4)$$

with e the void ratio. The compression modulus $M_v = 1/m_v$ also gives an indication of soil compressibility. The higher the M_v value, the less compressible the soil.

The compression characteristics of overconsolidated soil are demonstrated by the rebound (also known as unloading, decompression, swelling) and recompression curves. If recompression surpasses 1 MPa (the previous maximum stress), a straight line parallel to the already existing one will be obtained. The recompression curve indicates a clay that is overconsolidated and much less compressible than normally consolidated clays. The rebound is characteristic of the elastic deformation of the soil, whereas the difference between original and rebound height is indicative of the plastic deformation

of the soil. Elastic deformation is reversible and is primarily caused by bending and distortion of the solid matrix, whereas reorientation and fracture of the solid particles account for plastic deformation.

Recompression curves typically occur in preconsolidated soils, which are soils once subjected to a stress exceeding the present overburden pressure. Removal of that overburden by erosion, melting, lowering of the water table, or excavation leaves a soil preconsolidated. Most undisturbed soils are preconsolidated to some extent. This fact is extremely important in foundation engineering because such a soil will not settle appreciably until the stress imposed exceeds the preconsolidated stress (Sowers 1979). An unconsolidated soil with a low C_v can be preloaded with fill if normal consolidation is expected to last until after completion of the structure.

The coefficient of consolidation increases with increased permeability and decreased compressibility and is also inversely proportional to the specific weight of the diffusing fluid. Consequently,

$$k = C_v \lambda_w m_v. \quad (5)$$

1. Hackroy Series. The soil studied is a mixture of a typical profile of a Hackroy series, consisting of a loam, clay loam, and clay obtained from the Experimental Engineering Waste Burial Facility in Los Alamos, New Mexico.

The specimen dimensions: 100 mm X 100 mm X 25.5 mm

Moisture ratio by mass: 0.348

Mass of dry soil: 341 g

Particle density: 2.50 Mg m^{-3} (measured)

Initial void ratio: $0.348 \times 2.5 = 0.87$

Porosity: $0.87/1.87 = 0.465$

Bulk density (dry): $2.5/1.87 = 1.337 \text{ Mg m}^{-3}$

Moisture ratio by volume: $0.348 \times 1.337 = 0.465$

Saturated unit weight: $(2.5 + 0.87)/1.87 = 1.802 \text{ Mg m}^{-3}$

Volume: $341/1.337 = 255 \text{ cm}^3$

Height of sample: 25.5 mm

Liquid limit: 30% $\rightarrow C_c = 0.14$ (calculated compression index)
 Plasticity index: 5-10.

Both m_v and a_v and the computed hydraulic conductivity are seen to decrease with increasing stress (Table IV). The stress vs. void ratio graph, with log stress as the abscissa and void ratio as the ordinate, approximates a straight line. The compression index, C_c , is the slope of the straight line, where

$$e = -C_c \log \sigma / \sigma_0. \quad (6)$$

The index, C_c , is equal to 0.14345 above 60 kPa, where the line is straight (the higher C_c , the higher the compressibility of the material). The consolidation characteristics of a normally consolidated soil are depicted in the straight-line portion of the curve in Fig. 2. The swelling index is equal to 0.01826 or 13% of the compression index.

TABLE IV

C_c , m_v , AND k AS A FUNCTION OF STRESS FOR HACKROY SERIES

σ (kPa)	C_c ($10^{-6} m^2 s^{-1}$)	m_v ($10^{-7} Pa^{-1}$)	M_v (MPa)	a_v ($10^{-7} Pa^{-1}$)	k ($10^{-9} ms^{-1}$)
60	1.40	7.63	1.31	13.72	10.70
120	1.33	3.37	2.97	5.89	4.49
250	1.47	1.87	5.35	3.19	2.75
500	1.28	1.27	7.87	2.11	1.62
1000	1.25	1.27	7.87	2.05	1.60

The recompression curve follows a path more or less parallel to the rebound curve until the preconsolidation stress of 1 MPa is reached. Beyond the preconsolidation point, a fast acceleration in void-ratio decrease takes place, and the recompression curve merges with the virgin curve. The virgin, rebound, and recompression values at specific stresses are indicated in Table V and plotted in Fig. 2. It is clear from the graph that most of the deformation is plastic. This is to be expected considering the magnitudes of the contact pressures involved and the modulus of elasticity of soil grains, which is on the order of 20 GPa. Through regression analysis, we are able to determine that the best fit existing between hydraulic conductivity, k , as

Fig. 2. Virgin, rebound, and recompression curves for the Hackroy series soil.

dependent variable and void ratio, E , as independent variable is $k = 2 \cdot 10^{-12} e^{10.84e}$ with k in ms^{-1} . The coefficient of correlation is better than 0.94. This enables us to estimate the hydraulic conductivity for a nonconsolidated sample ($E = 0.87$) as being equal to $2.5 \cdot 10^{-8} \text{ ms}^{-1}$, or about 60 times smaller than that of crushed tuff.

2. Crushed Bandelier Tuff. Crushed Bandelier tuff has a grain-size distribution close to that of a sandy silt.

The specimen dimensions: 100 mm X 100 mm X 26 mm

Mass of dry soil: 365 g

Moisture ratio by mass: 0.323

Particle density: 2.56 Mg m^{-3} (measured)

Initial void ratio: $0.323 \times 2.56 = 0.83$

Porosity: $0.83/1.83 = 0.453$

Dry bulk density: $2.56/1.83 = 1.40 \text{ Mg m}^{-3}$

Moisture ratio by volume: $0.323 \times 1.40 = 0.45$

Saturated unit weight: $(2.56 + 0.83)/1.83 = 1.85 \text{ Mg m}^{-3}$

Volume: $365/1.40 = 260 \text{ cm}^3$

Height of sample: 26 mm.

TABLE V

H, Δe , e , $\Delta H/H$ AS A FUNCTION OF STRESS FOR THE VIRGIN, REBOUND, AND RECOMPRESSION CURVES FOR HACKROY SERIES

σ (kPa)	H (mm)	Δe	e	$\Delta H/H$
10	-0.350	-0.02567	0.84433	-0.01373
20	-0.650	-0.04767	0.82233	-0.02549
60	-1.428	-0.10472	0.76528	-0.05600
120	-1.944	-0.14256	0.72744	-0.07624
250	-2.564	-0.18803	0.68197	-0.10055
500	-3.132	-0.22968	0.64032	-0.12282
1000	-3.818	-0.27999	0.59001	-0.14973
600	-3.756	-0.27544	0.59456	-0.14729
400	-3.710	-0.27207	0.59793	-0.14549
200	-3.642	-0.26708	0.60292	-0.14282
100	-3.568	-0.26165	0.60835	-0.13992
10	-3.320	-0.24347	0.62653	-0.13020
100	-3.358	-0.24625	0.62375	-0.13169
200	-3.381	-0.24792	0.62208	-0.13259
400	-3.438	-0.25214	0.61786	-0.13482
600	-3.546	-0.26006	0.60994	-0.13906
1000	-3.928	-0.28805	0.58195	-0.15404

During consolidation, the data yielded void ratio-log time curves concave upward from the start, indicating extremely fast consolidation. The point, t_{50} , indicating the time at which 50% of the consolidation is complete, was always passed before the first measurement could be taken (at about 0.05 min). For our specimen of 26-mm thickness, C_v will then be at least $346 \text{ m}^2/\text{year}$ or $1.1 \cdot 10^{-5} \text{ m}^2 \text{ s}^{-1}$. On the other hand, the hydraulic conductivity, as well as both m_v and a_v , decreases with increasing stress (Table VI). The compression index, C_c , is equal to 0.14635 above 60 kPa. The void ratio-stress curve is slightly convex upward. The swelling index, S_c , equal to 0.01567, is smaller than that of the Hackroy series and is 11% of the compression index of tuff.

The recompression curve follows a path almost identical to the rebound curve until the preconsolidation stress of 1 MPa is neared. Beyond 1 MPa, the recompression curve should merge with the virgin curve. The virgin, rebound, and recompression values at specific stresses are indicated in Table VII and plotted in Fig. 3. The elastic deformation is even less for crushed tuff than for the Hackroy series. This can be deduced from the lower swelling index and the lower recovery ratio of swelling vs. compression for tuff (0.11 vs. 0.13).

As can be seen, the compression modulus is much more variable in the case of crushed tuff than Hackroy series soil and increases fast with stress. Crushed tuff is a very compressible material at low stress but quickly becomes incompressible at higher stress (faster than the Hackroy series soil).

The settlement at a pressure of 1 MPa is, according to Jumikis (1968), equal to $\Delta H = m_v H \sigma$ or, because ΔH is known,

$$\bar{m}_v = \frac{\Delta H}{H \sigma} \quad (7)$$

$$\bar{M}_v = \frac{H \sigma}{\Delta H} = \frac{25.5 \text{ mm} \cdot 10^6 \text{ Pa}}{3.818 \text{ mm}} = 6.7 \text{ MPa}$$

for Hackroy series soil, and

$$\bar{M}_v = \frac{26 \text{ mm} \cdot 10^6 \text{ Pa}}{3.424 \text{ mm}} = 7.6 \text{ MPa}$$

for crushed tuff. Both can be considered fairly compressible materials because they have rather low values of $\bar{M}_v$ (Hackroy series soils more so than crushed tuff).

The best fit between hydraulic conductivity, k , and void ratio, E , was determined through regression analysis: $k \propto 5.51 \cdot 10^{-13} e^{15.53E}$, with k expressed in ms^{-1} and $r = 0.97$. It is obvious that the values for the hydraulic conductivity are underestimated at all pressures because of an arbitrarily low choice of C_v .

At a porosity of 0.4 ($e = 0.67$), k would be equal to $1.81 \cdot 10^{-8} \text{ ms}^{-1}$. This is underestimating the measured hydraulic conductivity by a factor of ~ 80 . The relationship now becomes $k = 4.37 \cdot 10^{-11} e^{15.53E}$. A more correct C_v of $8.7 \cdot 10^{-4} \text{ s}^{-1}$ can now be estimated from the intrinsic relationship between hydraulic conductivity and coefficient of consolidation. Because k was known, a more direct approach could have been taken by using the formula expressing C_v as a function of k and computing C_v directly instead of trying to measure it. Our work also shows that only a static load better than 250 kPa can match the void ratio obtained under dynamic loading in our field experiments.

TABLE VI

 C_v , m_v , AND k AS A FUNCTION OF STRESS FOR CRUSHED TUFF

σ (kPa)	C_v ($m^2 s^{-1}$)	m_v ($10^{-8} Pa^{-1}$)	M_v (MPa)	a_v ($10^{-8} Pa^{-1}$)	k ($10^{-7} ms^{-1}$)
60	$8.7 \cdot 10^{-4}$	112.0	0.89	200.0	97.44
120	$8.7 \cdot 10^{-4}$	32.8	3.05	57.4	28.54
250	$8.7 \cdot 10^{-4}$	18.0	5.56	30.8	15.66
500	$8.7 \cdot 10^{-4}$	9.78	10.22	16.3	8.51
1000	$8.7 \cdot 10^{-4}$	6.03	16.58	9.75	5.25

3. Bentonite/Tuff Mix. The permeability of waste disposal facility liners and caps (i.e., moisture barriers), is important in geotechnical engineering. Permeability is the dominant parameter in the design and implementation of waste disposal facilities. Clay is prominent among the materials usually considered to line or cap disposal pits. Foremost among the problems connected with the use of clays is cracking during periods of desiccation, although both the Environmental Protection Agency (EPA) and the Nuclear Regulatory Commission (NRC) seem to feel that clays, as barriers to water leachate migration and inflow of water, are the principal materials to be considered as liners and caps in waste disposal facilities. Clays and soils, in general, also offer by far the longest service life of any liner material.

Use of clay mixes instead of pure clays may be warranted but not solely on the basis of economics; mechanical benefits may even become overriding in mandating the use of mixing. In Los Alamos, New Mexico, the use of local tuff (texture of sandy silt,* Abrahams 1963) with low amounts of bentonite appeared to be very promising in greatly decreasing hydraulic conductivity without showing any of the mechanical impairments of clays. Saturated Na-bentonite absorbs water up to 5 times its own mass to form a gel up to 15 times its own dry volume. Besides being less expensive, a liner or cap, consisting of a

*Sandy Silt: an unconsolidated sediment containing 10-50% sand and having a ratio of silt to clay greater than 2:1 (Folk, 1954).

TABLE VII

H, Δe , e , $\Delta H/H$ AS A FUNCTION OF STRESS FOR THE VIRGIN,
REBOUND, AND RECOMPRESSION CURVES FOR CRUSHED TUFF

σ (kPa)	H (mm)	Δe	e	$\Delta H/H$
60	-0.874	-0.06153	0.76847	-0.03362
120	-1.386	-0.09757	0.73243	-0.05331
250	-1.994	-0.14038	0.68962	-0.07669
500	-2.630	-0.18515	0.64485	-0.10115
1000	-3.414	-0.24035	0.58965	-0.13131
250	-3.264	-0.22979	0.60021	-0.12554
60	-3.142	-0.22120	0.60880	-0.12085
250	-3.242	-0.22824	0.60176	-0.12469
800	-3.374	-0.23753	0.59247	-0.12977
1000	-3.432	-0.24161	0.58839	-0.13200

Fig. 3. Virgin, rebound, and recompression curves for crushed tuff.

mix of the local medium and bentonite clay, would not visibly crack when desiccated. Cracking from desiccation can be further minimized by proper compaction. A low hydraulic conductivity, combined with acceptable mechanical characteristics, should be obtainable at some ideal mix of two materials, each possessing one or the other property. The objective of this research is to obtain the necessary data to assure that the use of such a mixture (e.g., sandy silt/bentonite) is effective in isolating waste from the environment. This research will also tell us the respective ratios at which ideal hydraulic and mechanical characteristics may be expected. Laboratory tests were performed at $22^{\circ}\text{C} \pm 2^{\circ}\text{C}$. The bentonite used in our experiments was 13-T and was obtained from the International Minerals and Chemicals Corporation, Des Plaines, Illinois.

One of the liabilities one faces when using Terzaghi's step-loaded method lies in the assumption that k , C_v , and m_v remain constant during that particular consolidation load step (Tavenas et al. 1979). Both C_v and mainly m_v show a tendency to decrease with increasing stress (Abeele 1984), and there is no reason to doubt that the behavior would be different as the void ratio is reduced during any particular consolidation step. Tavenas et al. (1983, Part I) show that the coefficient of consolidation may decrease by more than a factor of four during a particular clay-consolidation load step. The variability of the coefficient of consolidation with changing stress is not as drastic when the clay content in the soil decreases. No trend in C_v values was detected for any of the lower bentonite/crushed tuff ratios considered in this study. Therefore, the coefficients of consolidation computed for each stress were averaged and used as the mean coefficient of consolidation at a particular mixing ratio. Table VIII shows decreasing C_v values with increasing bentonite/tuff ratios, R , whereas the m_v values are more susceptible to changing stresses. The relationship between C_v and R can be written as

$$C_v = 0.06R^{-2},$$

with $r^2 = 0.99$ for $0.04 < R < 0.14$.

TABLE VIII
m, AND AVERAGE C, VALUES FOR VARYING
MIXING RATIOS AND STRESSES

R	$C_v(10^{-8}m^2s^{-1})$			
	$\sigma(kPa)$	$\bar{x}$	s	$m_v(10^{-6}Pa^{-1})$
0.04	100	41.6	6.18	34.5
	200			20.3
	400			12.2
	800			7.1
0.06	100	18.2	5.90	39.1
	200			30.2
	400			18.1
	800			10.0
0.075	100	11.6	4.84	66.3
	200			39.7
	400			22.7
	800			11.5
0.09	100	8.0	6.20	43.5
	200			37.2
	400			22.5
	800			14.6
0.11	100	5.3	3.29	40.3
	200			43.0
	400			23.4
	800			13.6
0.14	100	3.2	0.72	40.0
	200			43.0
	400			25.9
	800			13.4

To ensure that C_v is more or less constant during any load increment, the applied stress increase is never more than double the previous applied stress. Table IV indicates the relationship, based on Eq. (5), of the computed values of k to R for a particular stress. The computed hydraulic conductivity is expressed in $10^{-10} ms^{-1}$.

In view of the liabilities we encountered when deriving hydraulic conductivity by application of the consolidated method and considering the difficulties in determining the coefficient of consolidation at lower bentonite contents (<0.01), direct measurement of conductivity was also performed using the constant head method. At low bentonite ratios, the consolidation rate is too fast to be measured with any degree of accuracy.

During consolidation of pure crushed tuff (sandy silt), the void ratio-log time curves were concave upward from the start, indicating extremely fast consolidation and subsequent low degree of accuracy. The coefficient of consolidation averaged $8.7 \cdot 10^{-4} \text{ m}^2 \text{ s}^{-1}$ for repetitive stresses of 50, 100, 200, 400, and 800 kPa. An analysis of variance detected no trend in C_v with increasing stress because of the high values of the standard deviations for C_v at 0% bentonite. The conductivities obtained using the constant head method on uncompacted tuff (0 kPa) with low bentonite ratios (0-0.04) are in general agreement with the results obtained using the consolidation method for higher bentonite ratios (0.04-0.14). This is demonstrated in Fig. 4.

Table IX shows, for varying consolidation pressures, the close power relationship existing between R as independent variable and k as dependent variable (all r^2 are better than 0.99!). That trend is displayed linearly on a log-log plot in Fig. 5, with k decreasing with increasing clay fraction. Figure 5 contains only the results obtained using the consolidation method and R values varying from 0.04 to 0.14. The results obtained using the constant head method (0 kPa) are not shown in Fig. 5 because the regression equation showing the best fit is not a power function. The best fit for uncompacted mixes ($R = 0$ to 0.04) is $\log k = 5.065 - 94.298R$, with $r^2 = 0.982$ and k in $10^{-10} \text{ m s}^{-1}$. Figures 4 and 5 further demonstrate that hydraulic conductivity is a function not only of particle-size distribution of varying bentonite ratio, but also of void ratio (or applied stress). The conductivity of a porous material obviously decreases with void ratio, e , and e , in turn, decreases with increasing compaction pressure or stress, σ . The former is clearly shown in Table X and Fig. 6. Direct measurement of hydraulic conductivity using either the constant head method or the consolidation method produces a linear e versus $\log k$ relationship. A predictive empirical linear relationship between $\log k$ and e was first proposed by Taylor (1948):

$$\log k = \log k_0 - \frac{e_0 - e}{C_k} \quad (8)$$

where k_0 and e_0 may be in situ, remolded, or known preconsolidated values and C_k is a permeability change index. This type of relationship has become regarded as the most accurate way of expressing the variation of permeability with void ratio (Tavenas et al. 1983, Part II). The linear relationship between $\log k$ and e extends beyond strains of 20% for sandy silt/bentonite mixes, whereas Tavenas et al. (1983, Part II) limit the validity of this relationship to strains of less than 20% for most natural soft clays.

The interrelationship between $\log k$ and e is very important in the study of materials in caps or liners that can in any way influence the migration of pollutants from waste disposal pits. Indeed, in a homogenized material with uniform grain-size distribution (as the one likely to be used to line or cap a waste disposal pit), the porosity would be the only variable to influence the conductivity. The slope of the void ratio versus $\log k$ is defined as the permeability change index, C_k (Tavenas et al. 1983, Part II). Table XI seems to indicate that the permeability change index, C_k , and the compression index, C_c , are both increasing with increasing bentonite ratio. (The compression index, C_c , is the slope of the straight line where $e = -C_c \log \sigma / \sigma_0$). The values for the C_c/C_k ratio average 0.677, with $s = 0.035$ or a CV (coefficient of variation) of 5.1%.

A linear relationship can be established between C_k and C_c :

$$C_k = -0.053 + 1.706 C_c,$$

$$\text{with } r^2 = 0.983.$$

For sandy silt/bentonite mixes, C_k relates to e_0 as

$$C_k = -0.835 + 1.585e_0.$$

No apparent relationship seems to link the C_c/C_k ratio with the void ratio, e .

Fig. 4. Hydraulic conductivity as a function of bentonite/sandy silt ratios.

TABLE IX

**SATURATED HYDRAULIC CONDUCTIVITY (10^{-10} ms^{-1})
EXPRESSED AS A FUNCTION OF MIXING RATIOS
FOR DIFFERENT CONSOLIDATION PRESSURES**

Stress (kPa)	k	r^2
100	$0.021R^{-2.098}$	0.996
200	$0.024R^{-1.968}$	0.995
400	$0.015R^{-1.934}$	0.996
800	$0.009R^{-1.910}$	0.996

Fig. 5. Hydraulic conductivity as a function of bentonite/sandy silt ratios.

TABLE X

HYDRAULIC CONDUCTIVITY (in 10^{-12}ms^{-1})
AS A FUNCTION OF VOID RATIOS
FOR VARYING CLAY CONTENTS

R	log k	r^2
0.00	$2.722e + 4.499$	0.846
0.04	$4.893e + 0.406$	0.980
0.06	$3.871e + 0.132$	1.000
0.075	$2.519e + 0.918$	0.965
0.09	$2.274e + 0.814$	0.960
0.11	$2.107e + 0.713$	0.954
0.14	$1.916e + 0.559$	0.946

According to Tavenas et al. (1983, Part II), the condition for a constant C_v during consolidation may be written

$$\frac{1}{C_c} - \frac{1}{C_k} = \frac{1}{1 + e_0}. \quad (9)$$

As can be seen from Table XI, the left side of the equation exceeds the right side by a factor of two or three, thus failing once more to invalidate Terzaghi's assumption of the constancy of C_v during any particular loading step. This requirement had to be fulfilled for the consolidation method to be valid for the computation of k . However, our practical results using the consolidation method show a good compatibility with the results obtained using the constant head method or with the ones obtained by Daniel and Olson (1980) when using the same materials (tuff + bentonite). In fact, the results obtained by Daniel and Olson at 0 kPa are identical to our results at 400 kPa.

The predictive empirical linear relationship between $\log k$ and e first proposed by Taylor (1948) [Eq. (8)] allows us to compare the predicted (Taylor) vs. measured hydraulic conductivities (in 10^{-12} ms^{-1}) using the consolidation method. In no case did the discrepancy between the two methods amount to 3.5% (see Table XII).

The consolidation data were readily available because the computation of the hydraulic conductivity, in accordance with Terzaghi's theory, required measurement of the consolidation. Table XIII shows how the void ratio, e , varies as a function of stress, σ (or pressure), for different bentonite ratios, R . The goodness of fit of the data to the equation is expressed by the coefficient of determination r_e^2 ($\log \sigma$).

C_c and S_c are the consolidation and swelling indices obtained for different bentonite ratios. Figure 7 shows how both increase with increasing bentonite ratios. The S_c/C_c ratio averages 0.117, with a standard deviation of 0.018, or a coefficient of variation close to 16%. A linear relationship established between S_c and C_c yields $S_c = -0.005 + 0.140 C_c$ and $r^2 = 0.823$.

Fig. 6. Hydraulic conductivity as a function of void ratios for varying clay contents.

TABLE XI

COMPRESSION INDEX, PERMEABILITY CHANGE INDEX, AND DERIVED RELATIONSHIPS AS A FUNCTION OF CLAY RATIOS

R	C_c	C_k	C_c/C_k	$(1/C_c - 1/C_k)$	$1/(1+e_0)$
0.04	0.145	0.200	0.722	1.926	0.569
0.06	0.210	0.304	0.692	1.463	0.571
0.075	0.259	0.383	0.676	1.251	0.581
0.09	0.280	0.401	0.697	1.084	0.564
0.11	0.292	0.453	0.646	1.213	0.557
0.14	0.311	0.494	0.629	1.196	0.542

TABLE XII
PREDICTED (TAYLOR) AND MEASURED HYDRAULIC CONDUCTIVITIES
(in 10^{-12} ms^{-1}) AT 800 kPa
FOR VARYING CLAY RATIOS

<u>R</u>	<u>k</u> <u>Predicted</u>	<u>k</u> <u>Measured</u>	<u>$K_m - K_p$</u> <u>k_m</u>
0.04	2.613	2.636	0.009
0.06	2.270	2.299	0.013
0.075	2.081	2.114	0.016
0.09	1.896	1.964	0.035
0.11	1.759	1.799	0.022
0.14	1.626	1.602	-0.015

B. Consolidated, Drained (CD) Shear Test

When soil interfaces or surfaces are not horizontal, gravity will tend to slump a given soil mass downward. If an external force, static or dynamic in nature, joins with gravity, the shear stress along a soil interface or crack or any potential slip surface may cause rupture and subsequent movement of a given soil mass. This is the reason that shear strength in rocks and soils should always be evaluated before being submitted to a shear stress resulting from slopes created by excavations and aggravated by additional stresses (water movement and static or dynamic loads contrived by nature or man).

Negative stress induced by capillary tension will be at the origin of increased soil shear strength. Capillary tension is the driving force that enables moist sand to maintain a molded or cut shape. Thin water films with small meniscus radii develop high tensile stresses in the moisture wedges that hold soil particles in rigid contact. Fine sands and silts above a water table owe their strength to capillary tension and the resulting effective stresses in the granular structure. A point of maximum stress exists as a function of moisture content for a particular soil. In that case, any drying or wetting away from that optimum moisture content will mean a decrease in maximum shear strength. The components of shear strength are friction and

TABLE XIII

**CONSOLIDATION AND SWELLING OF
BENTONITE/SANDY SILT MIXES**

R	e	$r^2 e(\log \sigma)$	C_c	S_c	S_c/C_c
0.00	$0.915 - 0.129 \log \sigma$	0.993	0.129	0.016	0.108
0.04	$1.047 - 0.145 \log \sigma$	0.994	0.145	0.018	0.121
0.06	$1.171 - 0.210 \log \sigma$	0.996	0.210	0.021	0.101
0.075	$1.240 - 0.259 \log \sigma$	0.999	0.259	0.025	0.096
0.09	$1.333 - 0.280 \log \sigma$	0.990	0.280	0.033	0.119
0.11	$1.381 - 0.292 \log \sigma$	0.997	0.292	0.035	0.121
0.14	$1.465 - 0.311 \log \sigma$	0.998	0.311	0.047	0.151

Fig. 7. Consolidation and swelling indices as a function of bentonite/sandy silt ratios.

cohesion. The friction component is primarily affected by mechanical factors, whereas physicochemical factors affect the cohesion component. Cohesion is dependent on attractive forces at work in clay particle interactions. Water plays an important role in determining the magnitude of the cohesion component because it affects the distance between soil particles and, consequently, the attractive forces associated with air/water menisci (Baver et al. 1972). For any granular material, the strength characteristics depend heavily on the dry unit mass to which it is compacted. A higher dry unit mass will correspond to a higher shear strength, all other parameters being equal. Changes in both dry unit mass and shear strength are influenced by the same independent variable, i.e., moisture content. A plot of dry density vs moisture content will indicate that compaction at any given energy level becomes more efficient as the moisture content increases toward an optimum level, beyond which the efficiency decreases.

The least expensive way to improve soil stabilization is precisely through compaction. Soil stabilization, in turn, means the improvement of several physical properties that, among other things, determine the shear strength of that soil. Besides an increase in shear strength, the other physical properties of a soil improved by compaction are the related increase in dry density and subsequent decreases in compressibility, permeability, and shrinkage (this last property is primarily applicable to montmorillonite). Adequate compaction of pit overburden will improve several desirable properties that are important for good waste management.

The soil failure mechanism involves the sliding of a soil mass relative to the main soil body. It is assumed that in such a case, the soil along the entire shear surface is at a failure state or that the maximum shear resistance of the soil has been matched by applied stresses. To prevent failure, no shear stresses should be applied to the soil that exceed a fractional value of the peak shear strength. That fractional value is obtained by dividing the peak shear strength by a safety factor (SF).

The purpose of a shear test is to obtain the peak shear strength, τ ; the angle of internal friction, ϕ ; and the apparent cohesion, c , of a soil. A direct shear test will also provide shear stress-deformation characteristics.

The shear strength is obtained from Coulomb's shear strength equation

$$\tau = \sigma_{n_{eff}} \tan \phi + c, \quad (10)$$

where the slope, $\tan \phi$, is termed the coefficient of internal friction and

$\sigma_{n_{eff}}$ is the effective normal stress. In a non-cohesive soil, $c = 0$ and

the shear strength $\tau = \sigma_{n_{eff}} \tan \phi$.

The friction in cohesive soils is less than in cohesionless soils because clays are platelike in arrangement and can easily be reoriented when under shear stress. On the other hand, the apparent cohesion in clays is considerably larger than in sands because of a larger specific surface and interaction of surface forces. The capillary system in clays is of much smaller diameter and, as such, significantly increases the magnitude of the apparent cohesion. Consequently the value ϕ may remain minimal for clays (near zero in case of an unconsolidated, undrained test), whereas the c component assumes a major role. If the sample is allowed to consolidate under the normal load before shearing and to drain during shearing, the test is termed a CD test. The unconsolidated, undrained (UU), or "quick" test will only reveal the apparent cohesion component, c . This test is only valuable for short-term stability problems concerning saturated, temporary earthworks. The shear strength can be obtained from a single measurement.

A higher degree of consolidation means an increase in soil strength, which is caused by increased density. The consolidation process may continue during shearing (no increases in pore water pressure). The displacement rate during shearing is determined from the consolidation rate so that the potential further increase in consolidation is not hampered by the capacity to drain (a function of hydraulic conductivity).

The consolidated, undrained (CU) test is applicable for shear failure computations of consolidated clay dams or other slopes subjected to rapid water drawdown.

Direct CD shear tests of the controlled strain type were performed at three or more normal stresses for each condition (preconsolidation level, moisture content, tuff vs soil). Each of the three or more resulting stress-strain graphs obtained for the three or more applied normal stresses show a peak shear stress. The peak shear strengths are then plotted as a function of the effective normal stresses. The shear strength is then expressed analytically in the Coulomb equation,

$$\tau = \sigma_{n_{eff}} \tan \phi + C.$$

Coulomb's equation shows that the shearing resistance is made up of the following two components:

1. Friction, increasing with normal stress ($\tau = \sigma_{n_{eff}}$) caused by the interlocking of particles. Sand is a good example of a frictional and cohesionless soil. The Coulomb failure envelope passes through the origin.

2. Cohesion, independent of normal stress. Coulomb's failure envelope is virtually horizontal if saturated clay is not allowed to consolidate before or drain during shearing.

In the tests involving Hackroy series soils, no sharp peak is apparent when plotting τ against horizontal displacement.

The volume decreased continuously during shearing, although in far lesser amounts if shearing was preceded by higher-level preconsolidation. In no case was there any dilatancy.

Saturated, unpreconsolidated Hackroy series soil has a shearing strength of

$$\tau = 25.89 + 0.621 \sigma_n \quad r^2 = 0.99948,$$

whereas saturated Hackroy series soil preconsolidated at 500 kPa has a shearing strength of

$$\tau = 33.17 + 0.618 \sigma_n \quad r^2 = 0.99914.$$

For crushed tuff, no distinct peak was apparent. Virtually no decrease in shear stress with increased displacement was noticed after the ultimate shear stress was attained. Unpreconsolidated crushed tuff decreases in volume upon shearing, a behavior reminiscent of loose sand. That behavior changes if the sample is preconsolidated, and dilatancy occurs only if the preconsolidated sample is sheared in a submerged shearbox.

For saturated, unpreconsolidated crushed tuff, moisture ratio by volume (MRV) = 0.453, and dry density (γ_d) = 1.40 Mg m^{-3} :

$$\tau = 8.72 + 0.73 \sigma_n \quad r^2 = 0.99770.$$

For saturated crushed tuff preconsolidated at 1 MPa, MRV = 0.349 and dry unit weight (γ_d) = 1.667 Mg m^{-3} :

$$\tau = 23.48 + 0.819 \sigma_n \quad r^2 = 0.99279.$$

In comparing Hackroy series soil with crushed tuff, it is immediately obvious that soils have a higher apparent cohesion, whereas tuff has a higher coefficient of internal friction. The angle of repose, representing the angle of internal friction of a granular material at its loosest state, can be calculated from Coulomb's envelope. It amounts to 38° for crushed tuff and 32° for Hackroy series soils (when cohesion is no factor, as when the soil is dry and remolded). The repose angle of crushed tuff, which is higher than the normally expected range (30° - 35°), is probably mainly due to a higher-than-average angularity, surface roughness, and grain-size distribution, all of which will tend to increase that angle of repose. However, as the internal friction angle is within the expected value range, the apparent cohesion is invariably higher than expected. (For more details on shear testing see Abeele, 1984.)

Table XIV shows how the average secant elasticity module of both Hackroy series soil (HSS) and crushed Bandelier tuff (CBT) vary with stress. The data are not 100% accurate because they were read from a direct shear test with $\sigma_{\max} = 500$ kPa instead of a triaxial test.

TABLE XIV
ELASTICITY MODULUS AS A
FUNCTION OF STRESS

<u>σ(kPa)</u>	<u>HSS</u> <u>E(MPa)</u>	<u>CBT</u> <u>E(MPa)</u>
100	21	22
200	20	22
300	17	21

C. Resistance to Penetration

Resistance to the penetration of a probing instrument is an integrated index of compaction, moisture content, and type of material involved (crushed tuff, various clays, sand, etc.). As a penetrometer enters the soil, it will encounter resistance to compression, some friction between soil and metal, and the shear resistance of the soil, which, as described above, involves both internal friction and cohesion (Baver et al. 1972).

1. Rod-Shaped Laboratory Penetrometer. If left to desiccate from a saturated state, the resistance to penetration increases in both tuff and soils (Tables XV and XVI). At very low moisture content (2%), the attraction between particles breaks down completely in tuff, whereas it continues to increase in the Hackroy series soils, reaching its maximum at the lowest moisture content. Tuff regains its completely loose state at 1% moisture content. This is quite similar to results obtained on sands where zero shear strengths are apparent when sands are either dry or saturated. A small cohesion is even observed in moist sand because of surface tension (Head 1982).

Just as the shear strength of crushed tuff at a given moisture content is very much a function of its dry density, so is the strength of undisturbed or solid tuff equally dependent on its density (Purtymun and Koopman 1965). For

several sites in the Los Alamos area, the influences of density (D) on crushing resistance (CR) can be expressed as

$$CR = -383 + 51.72 \ln D,$$

where the resistance to crushing is expressed in MPa and the bulk density in kg m^{-3} .

2. Dutch Cone Static Field Penetrometer. The shape of this instrument precludes the influence of penetration depth on penetration resistance. This can be regarded as a distinct advantage over the pocket or laboratory penetrometer. The disadvantage of the Dutch Cone penetrometer is its size, which limits its application to field experiments. Moisture contents were measured but not controlled. This penetrometer measures a complexity of soil conditions, varying from moisture content to soil gradation, density, friction, cohesion, etc.

According to Sowers (1979), the undrained strength can be roughly approximated by $C = P/N$ where P is the measured point resistance and N... embodies the shape of the device. Values for N range between 5 and 15 for the Dutch Cone penetrometer, depending on sensitivity of the soil (very sensitive soils require a low N value). Taking N arbitrarily equal to 10 yields an average cohesion of 122.25 kPa with $s = 33.66$ kPa for undisturbed, consolidated Hackroy series soil and 20.63 kPa with $s = 4.96$ kPa for disturbed soil.

D. Vane Shear Test

The cohesion component can be obtained in the field using the vane shear test. Because the friction component is not measured (this would imply the application of increasing normal stresses), cohesion can be determined from a single measurement. The vane shear test is capable of performing on undisturbed samples what the unconsolidated, undrained test achieves in the laboratory. In the vane shear test, the vane is driven into the soil to the desired depth and rotated. The torque for shearing is measured. The shear

TABLE XV

RESISTANCE TO PENETRATION
IN TUFF AFTER PUDDLING

H ₂ O (%)	RP (kPa)
28	0
25	15
20	300
19	420
10.5	430
8.5	440
6.5	450
6	460
2	50
1	15

TABLE XVI

RESISTANCE TO PENETRATION
IN SOIL AFTER REMOLDING

H ₂ O (%)	RP (kPa)
28	1
25	1
17	100
16	150
14	400
13	450
12	460
11	475
9.5	500
2	>500
1	>>500

area is theoretically equal to that of the cylinder formed by the shearing action of the blade edges. The theoretical relationship existing among blade dimensions, torque, T, and cohesion is

$$T = \frac{4\pi r^2 C}{1000} \left(\frac{H}{2} + \frac{r}{3} \right), \quad (11)$$

or

$$C = \frac{1000T}{4\pi r^2 \left(\frac{H}{2} + \frac{r}{3} \right)} \quad (12)$$

if C is expressed in kPa and the blade dimensions in mm. For the standard height to radius ratio of 4,

$$C = \frac{1000T}{28/3 \pi r^3}. \quad (13)$$

If $r = 10 \text{ mm}$ (Roctest's standard blade),

$$c = \frac{3 \cdot 10^3 T}{28 \cdot 10^3} = 0.0341T. \quad (14)$$

The advantage of the vane shear test is that the cohesion profile of an undisturbed soil can rapidly be obtained.

The remolded vane shear strength is determined in situ after the vane has been rotated a minimum of 10 times in undisturbed soil. Remolding is used to determine the soil's sensitivity, which is the ratio of undrained strengths (undisturbed/remolded) due to disturbance. After a period of rest, thixotropy will add strength to the remolded specimen.

Cohesion measurements of the Hackroy series soils are compared in Table XVII. Disturbed soil samples refer to those broken up by heavy machinery and moved to an experimental plot.

The measured sensitivity was 2.62, which is a low to medium sensitivity. The undisturbed, consolidated tests refer to the field vane shear testing of soils where heavy machinery and/or a high pile of cobbles had been deposited for a certain length of time while the upper soil layer was saturated.

TABLE XVII
COHESIVENESS OF HACKROY SERIES SOIL

	$\tau(\text{kPa})$		Sensitivity	
	$\bar{X}$	s	$\bar{X}$	s
Undisturbed, unconsolidated	44.50	6.67	2.62	0.09
I.D. remolded	17.00	1.41		
Undisturbed, consolidated	118.00	46.90		
Disturbed	17.33	4.68		
Shearbox	25.89			
Shearbox: consolidated	33.17			
500 kPa				

III. PROGNOSIS

A. Tests in Use

Aside from the tests performed in Los Alamos to study soil stability, a number of tests are more specifically intended to predict settlement in granular materials. These are the cone penetrometer test, discussed above, the plate-bearing test, and the standard penetration test. A short review of the two latter tests follows.

1. Plate-Bearing Test. This test consists of a series of incremental loads with simultaneous measurements of the corresponding settlements of the soil area under stress. Field settlement can be predicted as being inversely proportional to the ratio of field-to-plate radius or width. Corrections have to be made for deep uniform deposits because of an increase of the elasticity modulus with depth. Terzaghi and Peck (1974) propose the following correction for settlement prediction if a plate with a 0.3-m square is used:

$$\Delta H = \Delta H_0 \left(\frac{6.56B}{3.28B + 1} \right)^2, \quad (15)$$

where

ΔH = predicted settlement under pressure, p ,

ΔH_0 = settlement of 0.3-m-square test plate under pressure, p , and

B = width of waste trench in meters.

Resistance to settlement will depend significantly on the internal friction angle, ϕ , which in turn is strongly dependent on the relative density: a compacted soil will settle less than a loose soil. The elasticity modulus is the soil property that most significantly influences settlement under high pressure.

2. Standard Penetration Test. This is the most commonly used field penetration test and entails the determination of the numbers of blows, N , required to drive a given split spoon sampler a certain distance by dropping a

particular mass from a predetermined height. Peck et al. (1974) relate allowable net bearing pressure, p (in kPa), to settlement, ΔH (in mm), as

$$p = 0.41 N \Delta H. \quad (16)$$

It is well known that for a granular soil, the penetration resistance is strongly correlated with the relative density of the material under stress. The above formula will also have to be corrected for overburden pressure because it has been shown (Peck et al. 1974) that the standard penetration blow count increased with increasing effective overburden pressure. The correction factor is

$$C_f = 0.77 \log \frac{1915}{\sigma} \quad (17)$$

with σ in kPa.

As an example, let us assume that we want to determine the allowable load at a depth of 8 m in a sandy silt if the blow count is 30 blows per 0.30 m. The wet density is 1.6 and we want to limit the settlement to 100 mm.

Weight of wet soil: $1.6 \times 1000 \text{ kg m}^{-3} \times 9.81 \text{ m s}^{-2} = 15.7 \text{ kN m}^{-3}$.

Overburden pressure: $15.7 \text{ kN m}^{-3} \times 8 \text{ m} = 126 \text{ kPa}$.

$$C_f = 0.77 \log \frac{1915}{126} = 0.91.$$

The corrected blow count for overburden pressure is $0.91 \times 30 = 27$. The allowable load is $0.41 \times 27 \times 100 = 1107 \text{ kPa} = 1.1 \text{ MPa}$.

A pressure of 1.1 MPa will, consequently, bring about a settlement of 100 mm. We must remember, however, that any theoretical estimate of settlement is an approximation because soils are not strictly elastic, homogeneous, and isotropic. According to Lambe and Whitman (1979), the best estimates of settlement can be obtained by

1. using elastic theories to estimate stresses,
2. obtaining strains or elasticity moduli, and
3. relying upon experience to compensate for sample disturbance.

B. Settlement

Settlements causing damage have been categorized as (1) total settlement, (2) differential settlement, and (3) slope of settlement curve.

Work by Grant et al. (1974) seems to point out a correlation between total settlement, ΔH , and slope of the settlement curve, $d(\Delta H)/dx$, and also a correlation between differential settlement, δ , and the slope of the settlement curve, $d(\Delta H)/dx$. They relate as follows (Dunn et al. 1980):

For clay

$$\Delta H = 1200 \frac{d(\Delta H)}{dx} \quad (18)$$

$$\delta = 650 \frac{d(\Delta H)}{dx}, \quad (19)$$

For Sand

$$\Delta H = 600 \frac{d(\Delta H)}{dx} \quad (20)$$

$$\delta = 350 \frac{d(\Delta H)}{dx}. \quad (21)$$

Based on these correlations, it is conceivable to use any of the above as independent variables for the computation of any other two dependent variables to serve as settlement criteria. For example, if our allowable total settlement for the sandy silt in use remains 100 mm, the settlement slope should not exceed 0.17, whereas the differential settlement should remain below 58 mm.

Prediction of soil settlement would be a simple affair if the criteria of elasticity, homogeneity, and isotropicity were fully satisfied. This rarely being the case, the elastic theory only serves as a guide in settlement predictions, and, despite the fact that the elastic modulus generally increases with depth, it plays a key role in any settlement computation.

Suppose an 8-m depth of fill is placed over loose sandy silt, located high above the water table and having a unit weight of 13.7 kN m^{-3} . We are asked to predict the settlement of an underlying layer of 10 m of that sandy silt if the same material is used as backfill.

At mid-depth in the sand, the stress, $\sigma = 5 \text{ m} \times 13.7 \text{ kN m}^{-3} = 68.6 \text{ kPa}$.

The stress increase $\Delta\sigma = 8 \text{ m} \times 13.7 \text{ kN m}^{-3} = 110 \text{ kPa}$.

Final stress at mid-depth $\sigma_f = 178.6 \text{ kPa}$.

$$\text{Settlement } \Delta H = \frac{\Sigma H_o}{1 + e_o} \Delta e \quad (22)$$

or

$$\frac{10 \text{ m}}{1.83} (-0.0546) = 0.36 \text{ m.}$$

(This example is based on actual values measured in Los Alamos, New Mexico, using crushed tuff, which has the texture of a sandy silt and whose actual geotechnical characteristics are the ones used in the preceding example.)

Also,

$$\Delta H = \Sigma H_o \bar{m}_v \Delta \quad (23)$$

or

$$10 \text{ m} \times 26 \cdot 10^{-8} \text{ Pa}^{-1} \times 1.1 \cdot 10^5 \text{ Pa} = 0.29 \text{ m,}$$

or

$$\Delta H = \frac{\Sigma H_0}{1 + e_0} \bar{a}_v \Delta \sigma \quad (24)$$

or

$$\frac{10 \text{ m}}{1 + 0.83} \times 45 \cdot 10^{-8} \text{ Pa}^{-1} \times 1.1 \cdot 10^5 \text{ Pa} = 0.27 \text{ m},$$

$$\text{because } \Delta e = -C_c \log \frac{\sigma}{\sigma_0} \text{ and} \quad (25)$$

$$\Delta H = \frac{\Sigma H_0}{1 + e_0} C_c \log \frac{\sigma_f}{\sigma} \quad (26)$$

or

$$\frac{10 \text{ m}}{1 + 0.83} \times 0.14635 \log \frac{178.6 \text{ kPa}}{68.6 \text{ kPa}} = 0.33 \text{ m}.$$

The four methods yield $x = 0.31 \text{ m}$ and $s = 0.04 \text{ m}$. This indicates a remarkable agreement if one considers the fact that the average volume compressibility, $\bar{m}_v$, and the average coefficient of compressibility, $\bar{a}_v$, were calculated from the m_v and a_v values at $\tau = 120 \text{ kPa}$ and 250 kPa found in "Geotechnical Aspects of Hackroy Sandy Loam and Crushed Tuff" (Abeele 1984). The stress was computed for mid-depth because the average initial effective stress is identical to the initial stress at mid-depth (stress increases directly proportional to depth).

A refined method (Holtz and Kovacs 1981) will be described later. That method is handled as if the profile consists of several different compressible strata. The total settlement is then equal to the sum of settlements for each compressible stratum: $\Delta H_{\text{tot}} = \Delta H_1 + \Delta H_2 + \Delta H_3 + \dots$. No shortcut should be made by averaging estimated individual stratum settlements because each is likely to possess a very proper and different coefficient of consolidation; therefore, each stratum must be analyzed individually.

Sowers (1979) indicates also that analyses performed by Schmertmann show that 90% of the distortion settlement in sandy soils occurs within a depth of twice the width, B , of the loaded area, which, in the case of waste disposal

sites, could be quite deep. Deeper than 2B, there is very little settlement from any surface load because E , the elasticity modulus, increases with depth and confinement whereas the effects of any surface load decrease rapidly with depth.

Delayed compression of sandy soils is rarely observed because consolidation is immediate but occurs cumulatively during each loading.

Safety factors required for computation of settlement design depend on how accurately the soil condition and the nature and compaction state of the waste are known and how critical a settlement failure would be. The permissible amount of settlement depends on soil uniformity, subsequent settlement, and dimension of the waste site, and the safety factor could vary accordingly between 1.5 and 4. To compare settling behavior in a material with much slower consolidation, we mixed our sandy silt (crushed tuff) with 4% bentonite and predicted a settlement in 10 m of such a saturated mix provided the same material as in the previous case was used as backfill (8 m of backfill having a unit weight of 13.7 kN m^{-3}).

Properties of the slightly preconsolidated bentonite/sandy silt mix are

Initial void ratio: $e_0 = 0.757$;

Compression index: $C_c = 0.145$;

Coefficient of consolidation: $4.16 \times 10^{-7} \text{ m}^2 \text{ s}^{-1}$;

Unit weight: 16 kN m^{-3} .

The water table is well below the area to be considered.

1. Initial effective stresses are first computed

a. at 0 m: $\sigma_{(0)} = 0$;

b. at -5m: $\sigma_{(-5)} = 16 \text{ kN m}^{-3} \times 5 \text{ m} = 80 \text{ kPa}$;

c. at -10 m: $\sigma_{(-10)} = 16 \text{ kN m}^{-3} \times 10 \text{ m} = 160 \text{ kPa}$.

2. Stress increase due to backfill,

$$\Delta\sigma = 13.7 \text{ kN m}^{-3} \times 8 \text{ m} = 109.6 \text{ kPa}.$$

3. Final effective stress, σ ,

- a. at 0 m: = 109.6 kPa;
- b. at -5 m: = 189.6 kPa;
- c. at -10 m: = 269.6 kPa.

4. If we assume one-dimensional consolidation and a one-time load application, then settlement ΔH yields [Eq (26)]

$$\Delta H = \frac{\Sigma H_0}{1 + e_0} C_c \log \frac{\sigma'(-5)}{\sigma(-5)} = \frac{10 \text{ m}}{1 + 0.757} \times 0.145 \log \frac{189.6 \text{ kPa}}{80 \text{ kPa}} = 0.31 \text{ m}.$$

Just as in the previous case, Holtz's method yields lower results than does Lambe's. If, however, the total thickness of the layer under pressure is divided into thinner layers, the accuracy of the results will be improved. The settlement of each layer is then summed to obtain the total consolidation settlement. A settlement computation (Table XVIII) can be used. Suppose we divided each layer into thicknesses of 1 m each. The mid-depths are then, respectively, at d (in meters) with corresponding values of σ , σ' , σ'/σ , $\log \sigma' / \sigma$ and

$$\frac{\Sigma H_0}{1 + e_0} C_c = 0.0825 \text{ (constant)}. \quad (27)$$

In this case, $\Sigma \Delta H = 0.40 \text{ m}$, which is a more accurate result and matches Lambe's result more closely. We see that the settlement estimate increased by 29% using the method improved by Holtz. The total consolidation for a 4% bentonite/sandy silt mix would consequently be 4.9% according to Lambe and 4% according to Holtz. It is generally agreed that consolidation settlements can be predicted only within a range of 20% (Holtz and Kovacs 1981). The above two methods barely fall within that range.

To compute the time rate of settlement, we need the relationship between the percentage consolidation, U , and a "time factor," T_v . This was derived mathematically by Terzaghi. If we consider the bentonite/sandy silt to have single drainage, the value H_{dr} (thickness of soil under stress/drainage

outlets) is equal to 10 m. The coefficient of consolidation approximates $1 \text{ m}^2/\text{month}$. Based on the above, we can construct Table XIX.

We can see that the consolidation, which has been found to be almost instantaneous in sandy silt, has increased noticeably with the addition of only 4% by weight of bentonite.

C. Subsidence

The distinction between subsidence and settlement is not always apparent. For those who make that distinction, subsidence is a vertical earth movement that, rapid or slow, can take on catastrophic proportions. Slow subsidence is caused by reducing the neutral stress and increasing the effective stress, for example, by pumping water or oil and causing some kind of passive consolidation. This, in turn, causes the ground surface to sink selectively.

TABLE XVIII
SETTLEMENT COMPUTATIONS

$d(\text{m})^a$	$\sigma(\text{kPa})^b$	$\sigma'(\text{kPa})^c$	σ'/σ	$\log \sigma'/\sigma$	$\Delta H(\text{m})^d$
0.05	8	117.6	14.700	1.167	0.096
1.5	24	133.6	5.567	0.746	0.062
2.5	40	149.6	3.740	0.573	0.047
3.5	56	165.6	2.957	0.471	0.039
4.5	72	181.6	2.522	0.402	0.033
5.5	88	197.6	2.245	0.351	0.029
6.5	104	213.6	2.054	0.313	0.026
7.5	120	229.6	1.913	0.282	0.023
8.5	136	245.6	1.806	0.257	0.021
9.5	152	261.6	1.721	0.236	0.019

^a d = mid-depths.

^b σ = initial effective stress.

^c σ' = final effective stress.

^d ΔH = settlement per layer.

TABLE XIX
TIME RATE OF SETTLEMENT

U	T,	$\Delta H(m)$	t(months)
0.1	0.008	0.04	0.8
0.2	0.031	0.08	3
0.3	0.071	0.12	7
0.4	0.126	0.16	13
0.5	0.196	0.20	20
0.6	0.286	0.24	29
0.7	0.403	0.28	40
0.8	0.567	0.32	57
0.9	0.848	0.36	85
0.95	1.129	0.38	113
1.00	∞	0.40	∞

Rapid subsidence occurs in mining areas where cavities produced by cave-ins gradually or sometimes abruptly reach the surface. The soil layer bridging the cavity then collapses and slides vertically downward. Disintegration of waste materials in shallow land burial can have the same effect.

Bracing of any excavation is required to prevent the phenomenon known as "lost ground" (Sowers 1979), which occurs when surrounding soil is being squeezed into newly-formed excavations. This is usually noticed when the excavated volume of soil exceeds the volume of the excavation when finished. This will lead to subsidence of areas immediately surrounding excavated waste pits. Such events are particularly troublesome in soft clays and can be remedied only by careful bracing.

IV. REMEDIAL ACTION

Settlement can be prevented or at least curtailed by building earth embankments on top of unconsolidated soils before the final structure is to be emplaced. Building and subsequent removal of such embankments, which will have a reduction of void ratio as a result, are termed preloading. If, as in the case of a waste pit, the lateral extent of the preload is large in comparison to the thickness of compressible waste, one-dimensional strain

computations may be adopted. Preloading may be considered attractive only if the compressible material (for example, crushed tuff + waste products) drains rapidly if saturated, so that preloading time is relatively short. To obtain this, relatively thin layers with a low coefficient of consolidation will have to prevail, or thick layers with a high coefficient of consolidation will qualify. In other words, if the coefficient of consolidation is low, the drainage path should be short. A higher load or surcharge will of course shorten the consolidation time, and the pit contents will be compressed to a higher effective stress.

A second way to obtain settlement curtailment is through soil stabilization. Soil stabilization means the improvement of a soil property so as to remediate its geotechnical performance. Soil stabilization may be intended to increase the elasticity modulus, which in turn may imply an increased strength or decreased compressibility or both, or it may portend decreased permeability. Soil improvement may be temporary in intent or permanent. Lambe and Whitman (1979) classify soil improvement techniques according to the process entailed, material addition, or intended result. Based on the process involved, stabilization can be induced mechanically, chemically, electrically, or thermally, and each of these can be accomplished in several ways. Densification of soils or void ratio reduction is most commonly obtained through static and dynamic (also vibratory) compaction.

Densification by means of rollers is best for the upper layers of a subgrade, sand (which can be densified with rollers to a 1- or 2-m depth), or freshly placed soil layers.

Granular soils can also be effectively densified using vibratory rollers. A 60 kN roller operating at a frequency of 27.5 Hz results in the most efficient compaction around a 0.6-m depth, which is the greatest depth at which zero effective stress occurs during rebound under above-described circumstances (see Fig. 8, adapted from D'Appolonia et al. 1968). Densification depth will increase somewhat with the number of passes (see Fig. 9, adapted from D'Appolonia et al. 1968).

Another form of dynamic consolidation is achieved by dropping heavy steel masses (up to 40 tons) from heights of up to 40 m. This method, developed in Europe, was proven effective to depths of 20 m with settlements amounting to 15% of the total compacted backfill thickness. The 40-ton mass is lifted by crane and dropped according to a predetermined pattern for the entire site. This method, however, could lead to the collapse of waste containers and the nefarious effects related to such events ranging from release of radioactive gas to exposure to percolating water. To avoid this, the drop height should be selected so that the effectiveness of the compactive effort does not extend beyond the backfill. Such impact force would be sufficient to collapse soil bridges over voids between containers (Kahle and Rowlands 1981).

Pile driving is aimed at the densification of cohesionless soils. In this case, densification is produced by displacement of material equal to the buried pile and by accompanying vibratory effects.

In cohesive soils, preloading (or surcharging), or the use of rubber-tired rollers with tire pressures up to 1 MPa are usually the most effective (static loads). The extrusion of a viscous cement/sandy loam mix into the soil voids can lead to a form of compaction (or reduction of void ratio) known as grouting. In situ soil compaction occurs through radial compression.

Figure 10 is a representative model of the manner in which dry unit weight of sand changes as a function of applied acceleration in a laboratory vibration study (adapted from D'Appolonia et al. 1968). Peak density was obtained at 2 g acceleration, with sand being most sensitive at acceleration changes around 1 g. The densification process seemed to be independent of the vibration amplitude. A purely static load does little to densify sand unless the stress is high enough to crush the sand granules. Consequently, something specific to a vibratory motion must be at the origin of sand densification. It has been proposed that, at the point in each vibrational cycle where the downward acceleration of the vibrating table reaches 1 g or more, the vertical (static) stress within the soil is zero. Because sand is a porous, loose material that cannot bear tension, it is unable to follow the motion to which the vibrating table is submitted and undergoes free fall until mutual

Fig. 8. Contours of maximum vertical dynamic stress beneath vibratory roller.

Fig. 9. Densification by vibratory roller.

Fig. 10. Representative model obtained in laboratory vibration study.

impact of the sand granules occurs when the motion reverses direction. Free-fall is what seems to characterize densification because it is only worthwhile at accelerations equal to or greater than 1 g. It is as if the absence of stress allows the particles to break physical contact, and they are driven into positions of optimum density as the vibrating mass reverses direction. The absence of a stress period seems to be essential to the densification process.

Optimum density is the density that can be obtained through compaction at an optimum moisture content. The most prevalent compaction test is the dynamic compaction test, consisting of dropping a hammer of specified mass a given number of times from a particular height on the soil to be tested. If a soil is compacted according to constant values for mass, height and number of blows, and variable water contents, then plotting of moisture content versus dry density will show that an optimum value of dry density can be attained as a function of water content (water content will cause the dry density to peak and subsequently decrease). We see in Fig. 11 that a maximum dry density of

1.83 (17.85 kN m^{-3} unit weight) is reached at an optimum moisture content of 13% for a bentonite/sandy silt ratio of 0.02.

Figure 12 shows clearly that by decreasing the compactive effort, the maximum dry density lowers in value and the optimum water content increases (Lutton et al. 1979). Also, as the moisture content increases, the cause and effect relationship between compactive effort and dry density tends to decrease. The line connecting the points of maximum dry density (or optimum water content) seems to run more or less parallel to the saturation line ($s = 100\%$). It is immediately obvious from this graph that the saturation ratio decreases with decreasing dry unit weight if the water content by mass remains the same. This shows only that a lower dry unit weight corresponds to a higher void ratio or porosity.

Fig. 11. Compaction test.

Fig. 12. Dry density as a function of water content.

The decrease in void ratio can also have drastic consequences for the hydraulic conductivity, as formerly depicted in Fig. 6 where void ratio is plotted against hydraulic conductivity for different bentonite ratios. Lutton et al. (1979) show the same effect taking place on different materials (Fig. 13). They also show the effect of void ratio on the angle of internal friction, demonstrating that for any particular soil, a decrease in void ratio inevitably leads to soil stabilization because of a higher angle of internal friction (Fig. 14). It should be kept in mind that both a soil strength increase and a reduced permeability resulting from one form of compaction or another positively affect the integrity of a waste pit cover.

Although soils compacted over waste pits are generally relatively soft, one should strive, on a granular soil-like solid waste, for 90% of maximum dry density obtained by the 25-blow standard compaction test. Figure 15 (Lutton et al. 1979) shows how compaction curves vary with various soil types. S. Phillips et al. (1983) show that, by mixing styrofoam with silty sand in a ratio of 1:1, the coefficient of compressibility, a_v , which varied from $1.88 \times 10^{-7} \text{ Pa}^{-1}$ to $1.04 \times 10^{-7} \text{ Pa}^{-1}$ for corresponding stress intervals of 0 to 239

and 239 to 575 kPa for silty sand, adopted values varying from $1.52 \times 10^{-6} \text{ Pa}^{-1}$ to $2.17 \times 10^{-6} \text{ Pa}^{-1}$ for the mix. Thus, these researchers conclude that a one-order-of-magnitude change in a_v is realized as the composition is changed to 50% highly compactible material. The compression index, C_c , also changes by one order of magnitude.

Lowering of the water table or dewatering is probably the best known cause of massive settlement. When submerged, soil particles are subjected to buoyancy. Upon dewatering, the buoyancy is removed and the apparent increase in pressure results in consolidation, even though there is no increase in external load. In the case of crushed tuff, the ratio of dry tuff density (γ_d) and submerged tuff (γ_s) is equal to:

$$\gamma_d/\gamma_s = \frac{(1-n)G\gamma_w}{(1-n)(G-1)\gamma_w} \quad (28)$$

or

$$\frac{1.54}{0.94} = 1.64,$$

where n = porosity = 0.40 under static load of 250 kPa; G = specific density of tuff particles = 2.56; γ_w = density of water = 1; and $\gamma_d/\gamma_s = 1.64$, meaning that relative density of crushed tuff is approximately 1.6 times higher when the tuff is dry than when it is submerged. This ratio is valid for most soils and is the main reason for the consolidation and subsequent subsidence of Mexico City, where the rate of pumping causes the city to settle at a rate close to 2 mm per day. High pumping rates and the thickness of the bentonite layer, which is known to have a void ratio as high as 15, and massive monuments and skyscrapers are the causes of the "disappearance" of the city. The volcanic ash, at the origin of the bentonite clay, has a unit weight averaging only 6 kN m^{-3} and, consequently, is very compressible when loaded (as by dewatering).

If dewatering is desired, i.e., means other than mechanical (pumping), such as drains and electro-osmosis, can be used for the construction and maintenance of a waste pit.

Fig. 13. Permeability of materials as affected by void ratio.

Fig. 14. Relation of effective angle of internal friction to void ratio for various soil types.

Fig. 15. Compaction curves for various soil types.

Vertical drains can consist of sand or geotextiles and are generally used in conjunction with preloading to accelerate clay consolidation.

V. FIELD SUBSIDENCE EXPERIMENT

A. Test Plan

Subsidence cavities measured on actual burial trenches vary widely in both size and shape--from broad, shallow depressions to narrow pipes that may extend to the waste. Burial site surveys indicate that about 85% of the measured cavities are less than 2.75 m in diameter, and 95% are less than 4.25 m in diameter.

To stress the biobarrier, cavities of four sizes were created. There are two replicates of each and two control plots. The experiments are conducted in a trench 38 m long, 15 m wide, and 3 m deep. In the bottom of each 58-m² experimental plot we augered a 0.9-m-diameter hole to a depth necessary to equal the desired volume of the subsided cavity (1.4, 3.4, 6.4, and 11.5 m

deep). Over each of these drawholes was a 2.25-m^2 steel plate with a hinged trap door, which was fastened by explosive closures. One side of the drawholes was cut away flat to a depth of 1 m to allow the door to open fully. The entire trench was backfilled to a depth of 2.2 m with crushed tuff, screened to remove particles larger than 5 cm. The backfill was overlain by 0.9 m of cobble/gravel biobarrier material and soil. A layer of cesium chloride tracer was placed at the backfill/barrier interface. Alfalfa was planted uniformly on the surface.

When the explosive closures were released, the trap doors fell downward, allowing the backfill to drain into the drawholes and causing subsidence at the surface. Slow subsidence of the entire trench surface, resulting from continued stabilization of the backfill, should be observable throughout the duration of the experiment.

Plant root penetration is being monitored by routine sampling of plant leaves. Cesium concentrations in the leaves will be mapped as a function of time and location relative to the subsided cavities. Root penetration (if any) can be expected to occur first at the cavity rims--regions of maximum tensile stress and elongation.

At the end of the experiments, the plots will be excavated to measure the actual degree of root penetration through the barrier. At the same time, both the upper and lower surfaces of the biobarrier will be mapped to determine the physical effects of subsidence on the barrier and to correlate with the tracer data and root measurements.

B. Preliminary Results

The resistance to subsidence should be equal above all eight drawholes because the main parameters influencing subsidence are unchanged in the backfill overlying the eight drawholes. The uniform backfill thickness/drawhole diameter ratio (t/d) was high enough to prevent subsidence at any of the eight locations. For some time it looked as if subsidence would occur by accident (as it eventually does in a completely natural environment) or some method would have to be found to induce or enhance subsidence without

using disruptive mechanical means, which would leave a permanent imprint of "artificial" intervention.

From this experiment it is obvious that the crushed tuff and/or the soil have some cohesiveness, as was demonstrated in the laboratory (Abee 1984). The lab results also show that, even for crushed tuff, a higher degree of consolidation or compression is at the origin of an increase in soil strength. (It is well known that densification causes soil stabilization.) The bottom of the landfill, which is submitted to a pressure averaging 50 kPa, could consequently be fairly well stabilized when dry.

A completely cohesionless porous medium (Ottawa sand, for example) would have undergone immediate subsidence into the 0.9-m-diameter drawholes when the trapdoors were released. This was obviously not observed when the trapdoors, overlain by crushed tuff, were opened.

As stated earlier, the presence of excess water reduces the effective stress responsible for the friction between solids. Therefore, it was decided that by increasing the water content of the backfilling, the shear strength may decrease enough to cause failure or subsidence while preserving the "natural" setup. This action could in no way be considered totally undisturbing to the environment because it was suspected that the amount of water needed would far exceed the amount of water available through natural precipitation in Los Alamos. Figure 16 shows the average moisture content as a function of depth in a typical monitoring hole before addition of water.

Flooding of the area immediately overlying the drawholes caused subsidence in two 1.4-m deep holes, two 3.4-m-deep holes, two 6.4-m-deep holes, and one 11.5-m-deep hole. This is one hole more than was thought possible because it was speculated that two trapdoors had failed to open.

The shape of the subsidence holes was far from resembling an inverse cone with regular slope. Instead, it had, in most cases, a vertical wall where the cohesive materials are located (the Hackroy series soil), and extremely irregular angles where the diameter of the unstable moving material is large compared with the height of the slope (gravel and cobble in our case). The

Fig. 16. Moisture as a function of depth--Hole 401.

requirement that the diameter of the unstable moving material be small compared with the total slope is made to satisfy the demand for obtaining the angle of repose, which represents the angle of internal friction and/or maximum slope angle of a granular material at its loosest state. The diameter-to-length ratio of the slope is too high in the case of gravel and cobble and the compression is too high in the crushed tuff for the slope angle to be representative of the angle of repose. Cohesion prevents the Hackroy series soil from adopting an angle that would be indicative of what the angle of repose might be.

The volume of the cones is extremely difficult to compute for the two smaller ones but averages around 90-95% of the drawhole volume for the remaining five. These results are justifiable because pores created by rocks filling the drawholes will be at the origin of a lower bulk density in the drawhole and will correspond to a smaller conic volume at the surface.

Principles based on relationships between surface deformation and underground cavities can be applied to predict fundamental quantities such as maximum possible subsidence. Generalization of these empirical relationships can lead to calculation of complete deformation profiles, provided

1. the stratification is horizontal (soil, biobarrier, tuff);
2. the subsidence reached its final stage; and
3. the cavities are geometrically simple.

Because the above conditions are fulfilled, final deformation is characterized by the following facts:

1. The surface subsidence boundaries extend beyond the horizontal edges of the cavity.
2. Concurrent with subsidence, stress-producing horizontal displacements occur, whose magnitude depends on the subsidence slope. Those movements are larger than would be expected from the subsidence curvature.

3. The cylindrical nature of the cavity causes maximum subsidence over the center, where there is no horizontal movement, whereas the vertical and horizontal stresses and subsequent displacements should be symmetrically distributed over the subsidence area.

The vertical component, whose upper limit is defined as "maximum possible subsidence" is present only if the cavity has a minimum "critical area."

In case a critical area is present, the central maximum possible subsidence is coupled with zero curvature and strain (Fig. 17). Prediction of maximum subsidence is based on the fact that it is correlated to cavity thickness, or

$$S = at, \quad (29)$$

where a = subsidence factor.

If the displacements caused by any cavity on our plot are affected by displacements caused by neighboring cavities, then we would witness a superposition of surface displacements. Because this was not the case, we can assume that any cavity is unaffected (through distance) by the presence of any other.

Maximum subsidence is also dependent on the subsidence factor, which in turn depends on the depth of the cavity, its lateral dimensions, and stability of overlying soil layers. Because these three parameters are the same for all cavities, the only variable remaining in our plot is t . The subsidence factor would be very difficult to determine for our heterogeneous overburden, but one would expect it to decrease with increasing depth. The General Institute of Mining Surveying (1958) suggests

$$S = \frac{25 \text{ m}}{25 + \sqrt{h}} \cos a. \quad (30)$$

This formula does indeed point to a decrease of subsidence with depth of drawhole location.

The National Coal Board, Mining Department (1975), tried to predict maximum subsidence based on curves empirically derived from actual measured occurrences, which appear under certain conditions. However, those curves are not drawn for cavities of less than 10 m in diameter.

Fig. 17. Subsidence and surface movements (Brauner 1973).

C. Biointrusion

Statistical analyses of data from the short-term, small-scale biointrusion studies conducted in lysimeters (Hakonson et al. 1981) revealed that a trench cap design consisting of 60 cm of topsoil over 25 cm of gravel (2-cm diameter) over a 75-cm layer of cobble (75- to 13-cm diameter) effectively limited both plant root and burrowing animal intrusion into a simulated waste emplaced beneath the cap. Although the results from this initial screening experiment were encouraging, a number of additional questions remained concerning the long-term performance of a soil/rock intrusion-barrier cap design. Those questions were

- o How does the soil/rock cap design affect water balance, particularly percolation?
- o How does the soil/rock cap design perform at larger scale?
- o How does the soil/rock cap design perform over extended time?
- o How much subsidence can be permitted so that the effectiveness of the soil/rock intrusion-barrier cap design is maintained?

To address the question of intrusion barrier performance under various degrees of subsidence, the design and construction of the plot is described in detail in a previous section.

Evaluating the effectiveness of the soil/rock intrusion-barrier cap design under various degrees of subsidence was accomplished through the use of a tracer emplaced at the interface of the trench cap and underlying backfill. A total of 73 kg of CsCl was spread uniformly, in a thin layer, on the crushed tuff backfill before placement of the soil/rock trench cap. Because cesium is plant-available, time series analysis of the cesium content of vegetation samples can be used to indicate root penetration through the trench cap.

Although the entire plot area was seeded with a mixture of native grasses, the only plant that was successfully established on the plot was a common invader (or weed) of the genus *Euforbia*. Plant cover during the height of the growing season in 1983 was about 50%. The lack of success in establishing native grass cover stems from our decision not to supplement precipitation by irrigating the plot.

Vegetation sampling on each of the plots was begun in July 1983. Samples were oven dried and submitted for neutron activation analysis to determine cesium content. Cesium concentrations in excess of 1 ppm (background levels in plants are < 1 ppm) were considered indicative of root penetration to the cesium layer.

VI. SUBSIDENCE ESTIMATION

Phillips (1983) devised a method to estimate geomechanical subsidence. If we consider that the total drawhole volumes are 0.89 m^3 , 2.16 m^3 , 4.07 m^3 , and 7.32 m^3 , respectively, and that they all occurred at 3.1 m below grade, prediction for subsidence depths and diameters could be attempted.

Using the Subsidence Feature Estimation Curves from Phillips, the predicted maximum subsidence depths for the 0.89 m^3 , 2.16 m^3 , and 4.07 m^3 drawholes amount to 0.3 m, 0.6 m, and 0.8 m as compared with the measured values of 0.3 m, 0.8 m, and 1.3 m, respectively. Only the cavities of 2.16 m^3

and 4.07 m^3 were completely surveyed. The curves were derived from soil mechanics studies in idealized, noncohesive, isotropic, and homogeneous porous media, and the morphology of the voids used herein by Phillips are idealized, i.e., a cylinder equivalent in diameter and length. The closer the voids studies at Los Alamos resemble the idealized void, the better the match between the predicted and the measured maximum subsidence (the smaller void depth is the one that most closely matches the idealized void.)

Our subsidence feature appeared at the rim as a near vertical depression, which is slowly being transformed to a more shallow depression of larger diameter as the particulates form increasingly stable slopes.

According to Phillips' Subsidence Feature Estimation Curves, the estimated maximum subsidence diameters for the surveyed cavities of 2.16 m^3 and 4.07 m^3 are 4.75 m and 5.2 m, respectively. This compares to measured values of 3.7 m and 4.25 m.

It has to be realized, as was stated in the procedures' limitations, that the predictions were valid in idealized, noncohesive, isotropic, and homogenous porous media. This is obviously not the case in our experiment. Neither are most of the studied drawholes anything near an idealized void. This last point, more than anything else, is probably at the origin of the noted discrepancy.

VII. RESULTS AND DISCUSSION

It can be seen that, regardless of the fact that our drawholes have far from an idealized void or medium, the shape of our cavities is slowly approaching the one predicted by Phillips. Indeed, the depth of the cavity is decreasing as the diameter is increasing, mainly through factors such as erosion. (Phillips' depths are more shallow and his predicted diameters are larger than the ones measured to date.) Our depth measurements average between 133% and 163% of the predicted amount, while our diameters measure 70% and 82%. The actual shapes of the 2.16 m^3 and 4.07 m^3 cavities are depicted in Figs. 18-21. Figures 22-24 show the evolution of moisture content with time after the artificial addition of water for subsidence purposes. (See the

Appendix for a complete listing of the moisture data.) Throughout 1983 and 1984 no cesium was apparent over the control or subsided areas. The common invader plant of the genus *Euforbia* never exceeded 1 ppm in cesium content, leading one to believe that, whether or not the biobarrier lost some of its integrity in some cases, naturally occurring vegetation was unable to penetrate it. In 1985, however, failure of the biobarrier was general, due to an unusual precipitation pattern that during the first six months of the year was 250% of normal, leading to an unusually luxurious and forceful vegetation pattern. The location that showed the lowest concentrations of cesium was in the *Euforbia* growing at the center of the 4.07 m³ cavity, presumably because most of the cesium located below the biobarrier had dropped out of reach in the drawhole. The average cesium concentration in that cavity amounted to 23.4 ppm with a standard deviation of 36.4 ppm (not very homogeneous). Next in line was the 2.16 m³ cavity with an average cesium concentration of 123.5 ppm and a standard deviation of 52.6 ppm. The highest concentration was found in the *Euforbia* covering the control plot: 130 ppm. All this leads us to believe that, subsidence or no subsidence, failure of the biobarrier (and/or moisture barrier) was general, and subsidence only attenuated cesium uptake by putting it out of reach of the plants growing within the cavity. A true control would be one where subsidence would have occurred over a similar drawhole deprived of a biobarrier.

TRANSECT CAVITY 1

NOTE: 600 INDICATES SOIL SURFACE -
REFERENCE LEVEL IN cm

Fig. 18. 25-cm contour intervals of 4.07-m³ cavity.

TRANSECT CAVITY 2

NOTE: 500 INDICATES SOIL SURFACE -
REFERENCE LEVEL IN cm

Fig. 19. 25-cm contour intervals of 4.07-m³ cavity.

TRANSECT CAVITY 3

NOTE: 600 INDICATES SOIL SURFACE -
REFERENCE LEVEL IN cm

Fig. 20. 25-cm contour intervals of 2.16-m³ cavity.

TRANSECT CAVITY 4

NOTE: 500 INDICATES SOIL SURFACE -
REFERENCE LEVEL IN cm

Fig. 21. 25-cm contour intervals of 2.16-m³ cavity.

No. 401 40-cm to 160-cm Depth

Fig. 22. Moisture content after subsidence.

No. 401 180-cm to 240-cm Depth

Fig. 23. Moisture content after subsidence.

No. 401 260-cm to 340-cm Depth

Fig. 24. Moisture content after subsidence.

ACKNOWLEDGMENTS

Thanks are due to C. Lujan and G. Langhorst for computer graphics and to J. Steger and J. Nyhan for guidance.

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APPENDIX

VOLUMETRIC WATER CONTENT DATA BASE FOR SUBSIDENCE PLOTS

----- Subsidence Plot Tube Number - 401-----

Sampling Date	Depth From Surface (cm)	Volumetric Moisture Content (%)
07JAN84	40	10.1
07JAN84	60	10.0
07JAN84	100	10.3
07JAN84	140	10.4
07JAN84	160	10.4
07JAN84	180	11.3
07JAN84	200	12.8
07JAN84	220	14.5
07JAN84	240	15.6
07JAN84	260	16.9
07JAN84	280	17.4
07JAN84	300	9.0
07JAN84	320	3.5
07JAN84	340	7.6
07JAN84	360	21.9
23AUG84	40	9.0
23AUG84	60	8.1
23AUG84	100	8.5
23AUG84	140	9.9
23AUG84	160	11.9
23AUG84	180	12.7
23AUG84	200	14.0
23AUG84	220	15.3
23AUG84	240	16.1
23AUG84	260	8.5
23AUG84	280	4.1
23AUG84	300	3.9
23AUG84	320	4.4
23AUG84	340	7.4
23AUG84	360	19.1
27SEP84	40	8.3
27SEP84	60	7.9
27SEP84	100	8.5
27SEP84	140	9.3
27SEP84	160	10.2
27SEP84	180	10.9
27SEP84	200	11.5
27SEP84	220	12.4
27SEP84	240	13.7
27SEP84	260	14.0
27SEP84	280	15.5
27SEP84	300	15.6
27SEP84	320	7.0
27SEP84	340	3.3
27SEP84	360	5.6
31OCT84	40	9.3

31OCT84	60	9.1
31OCT84	100	10.3
31OCT84	140	10.6
31OCT84	160	10.9
31OCT84	180	11.4
31OCT84	200	12.6
31OCT84	220	13.2
31OCT84	240	14.2
31OCT84	260	15.3
31OCT84	280	16.0
31OCT84	300	10.6
31OCT84	320	3.7
31OCT84	340	6.3
31OCT84	360	21.7
30NOV84	40	10.4
30NOV84	60	10.5
30NOV84	100	11.2
30NOV84	140	11.1
30NOV84	160	11.5
30NOV84	180	11.8
30NOV84	200	12.8
30NOV84	220	13.6
30NOV84	240	14.2
30NOV84	260	15.1
30NOV84	280	15.4
30NOV84	300	7.4
30NOV84	320	3.7
30NOV84	340	8.5
30NOV84	360	21.6
13DEC84	40	10.2
13DEC84	60	10.3
13DEC84	100	10.7
13DEC84	140	10.6
13DEC84	160	11.1
13DEC84	180	11.3
13DEC84	200	12.1
13DEC84	220	12.7
13DEC84	240	13.2
13DEC84	260	14.2
13DEC84	280	14.5
13DEC84	300	7.0
13DEC84	340	3.4
13DEC84	360	7.2
20DEC84	40	10.1
20DEC84	60	10.1
20DEC84	100	10.6
20DEC84	140	10.7
20DEC84	160	10.9
20DEC84	180	11.2
20DEC84	200	12.0
20DEC84	220	12.7
20DEC84	240	13.2
20DEC84	260	13.8
20DEC84	280	14.1

20DEC84	300	7.5
20DEC84	320	3.4
20DEC84	340	7.4
20DEC84	360	22.1
07JAN85	40	10.1
07JAN85	60	10.0
07JAN85	100	10.3
07JAN85	140	10.4
07JAN85	160	10.4
07JAN85	180	11.3
07JAN85	200	12.6
07JAN85	220	14.5
07JAN85	240	15.6
07JAN85	260	16.9
07JAN85	280	17.4
07JAN85	300	9.0
07JAN85	320	3.5
07JAN85	340	7.6
07JAN85	360	21.9
11FEB85	40	10.4
11FEB85	60	10.6
11FEB85	100	11.2
11FEB85	140	11.7
11FEB85	160	11.9
11FEB85	180	12.1
11FEB85	200	13.6
11FEB85	220	14.5
11FEB85	240	15.9
11FEB85	260	17.2
11FEB85	280	18.4
11FEB85	300	8.3
11FEB85	320	4.0
11FEB85	340	8.7
11FEB85	360	27.9
15MAR85	40	12.9
15MAR85	60	13.3
15MAR85	100	14.4
15MAR85	140	14.3
15MAR85	160	15.1
15MAR85	180	15.6
15MAR85	200	16.7
15MAR85	220	19.0
15MAR85	240	21.1
15MAR85	260	22.5
15MAR85	280	23.4
15MAR85	300	11.5
15MAR85	320	4.4
15MAR85	340	10.0
15MAR85	360	27.6
20MAR85	40	13.1
20MAR85	60	13.7
20MAR85	100	15.7
20MAR85	140	16.3
20MAR85	160	17.4

20MAR85	180	18.3
20MAR85	200	19.5
20MAR85	220	20.9
20MAR85	240	22.1
20MAR85	260	22.6
20MAR85	280	22.8
20MAR85	300	9.3
20MAR85	320	4.4
20MAR85	340	10.7
20MAR85	360	28.2
27MAR85	40	16.3
27MAR85	60	16.7
27MAR85	100	17.6
27MAR85	140	17.6
27MAR85	160	17.8
27MAR85	180	18.1
27MAR85	200	19.0
27MAR85	220	20.7
27MAR85	240	20.3
27MAR85	260	20.7
27MAR85	280	20.3
27MAR85	300	9.5
27MAR85	320	4.0
27MAR85	340	9.8
27MAR85	360	25.6
02APR85	40	18.7
02APR85	60	17.7
02APR85	100	18.2
02APR85	140	18.4
02APR85	160	18.5
02APR85	180	19.2
02APR85	200	19.9
02APR85	220	21.5
02APR85	240	22.4
02APR85	260	23.0
02APR85	280	23.0
02APR85	300	11.7
02APR85	320	4.8
02APR85	340	11.1
02APR85	360	29.1
25APR85	40	16.8
25APR85	60	16.0
25APR85	100	16.6
25APR85	140	16.6
25APR85	160	16.8
25APR85	180	16.5
25APR85	200	17.3
25APR85	220	18.3
25APR85	240	18.3
25APR85	260	18.2
25APR85	280	17.9
25APR85	300	8.6
25APR85	320	3.9
25APR85	340	8.8

25APR85	360	20.7
16MAY85	40	17.5
16MAY85	60	16.4
16MAY85	100	17.2
16MAY85	140	16.7
16MAY85	160	15.6
16MAY85	180	16.9
16MAY85	200	17.7
16MAY85	220	18.6
16MAY85	240	18.4
16MAY85	260	18.8
16MAY85	280	18.4
16MAY85	300	8.9
16MAY85	320	3.8
16MAY85	340	8.2
16MAY85	360	19.1
21MAY85	40	17.5
21MAY85	60	16.4
21MAY85	100	16.6
21MAY85	140	16.3
21MAY85	160	16.2
21MAY85	180	16.4
21MAY85	200	16.8
21MAY85	220	18.0
21MAY85	240	18.4
21MAY85	260	18.4
21MAY85	280	18.0
21MAY85	300	8.0
21MAY85	320	3.8
21MAY85	340	8.6
21MAY85	360	22.9
28MAY85	40	16.8
28MAY85	60	16.1
28MAY85	100	16.2
28MAY85	140	16.1
28MAY85	160	16.0
28MAY85	180	16.1
28MAY85	200	16.7
28MAY85	220	17.9
28MAY85	240	18.2
28MAY85	260	18.1
28MAY85	280	17.5
28MAY85	300	7.8
28MAY85	320	4.0
28MAY85	340	9.1
28MAY85	360	21.6
11JUN85	40	16.7
11JUN85	60	15.7
11JUN85	100	16.0
11JUN85	140	15.7
11JUN85	160	15.8
11JUN85	180	15.3
11JUN85	200	16.3
11JUN85	220	17.6

11JUN85	240	17.5
11JUN85	260	17.3
11JUN85	280	16.8
11JUN85	300	7.8
11JUN85	320	3.6
11JUN85	340	6.2
11JUN85	360	12.5
03JUL85	40	16.7
08JUL85	40	16.2
08JUL85	60	15.3
08JUL85	100	15.1
08JUL85	140	15.3
08JUL85	160	14.9
08JUL85	180	15.5
08JUL85	200	15.5
08JUL85	220	16.4
08JUL85	240	16.5
08JUL85	260	16.7
08JUL85	280	15.8
08JUL85	300	7.7
08JUL85	320	3.4
08JUL85	340	5.7
08JUL85	360	9.0
16JUL85	40	15.2
16JUL85	60	14.4
16JUL85	100	14.6
16JUL85	140	14.5
16JUL85	160	14.7
16JUL85	180	14.5
16JUL85	200	15.3
16JUL85	220	15.9
16JUL85	240	16.7
16JUL85	260	17.4
16JUL85	280	18.3
16JUL85	300	13.1
16JUL85	340	6.3
16JUL85	360	21.0
01AUG85	40	15.3
01AUG85	60	15.0
01AUG85	100	14.6
01AUG85	140	14.6
01AUG85	160	14.4
01AUG85	180	15.2
01AUG85	200	15.3
01AUG85	220	16.4
01AUG85	240	16.6
01AUG85	260	16.5
01AUG85	280	16.1
01AUG85	300	8.2
01AUG85	320	3.4
01AUG85	340	5.6
01AUG85	360	10.3
12AUG85	40	15.3
12AUG85	60	14.5

12AUG85	100	14.8
12AUG85	140	15.1
12AUG85	160	14.6
12AUG85	180	14.9
12AUG85	200	16.3
12AUG85	220	17.0
12AUG85	240	17.9
12AUG85	260	18.3
12AUG85	280	18.8
12AUG85	300	13.6
12AUG85	320	3.5
12AUG85	340	6.1
12AUG85	360	20.6

----- Subsidence Plot Tube Number - 402 -----

Sampling Date	Depth From Surface (cm)	Volumetric Moisture Content (%) ?
07JAN84	40	12.1
07JAN84	60	11.9
07JAN84	100	11.8
07JAN84	140	11.7
07JAN84	160	12.5
07JAN84	180	13.4
07JAN84	200	14.2
07JAN84	220	15.3
07JAN84	240	16.7
07JAN84	260	17.6
07JAN84	280	12.3
07JAN84	300	4.1
07JAN84	320	3.9
07JAN84	340	15.0
07JAN84	360	19.1
23AUG84	40	14.2
23AUG84	60	14.0
23AUG84	100	14.2
23AUG84	140	13.9
23AUG84	160	14.8
23AUG84	180	15.7
23AUG84	200	16.8
23AUG84	220	17.1
23AUG84	240	17.3
23AUG84	260	4.5
23AUG84	280	3.9
23AUG84	300	4.0
23AUG84	320	5.3
23AUG84	340	15.9
23AUG84	360	8.4
27SEP84	40	13.0
27SEP84	60	12.5
27SEP84	100	12.3

27SEP84	140	12.5
27SEP84	160	12.8
27SEP84	180	14.1
27SEP84	200	14.3
27SEP84	220	15.1
27SEP84	240	15.1
27SEP84	260	15.2
27SEP84	280	10.5
27SEP84	300	4.0
27SEP84	320	3.6
27SEP84	340	9.8
27SEP84	360	8.8
31OCT84	40	13.0
31OCT84	60	12.9
31OCT84	100	12.8
31OCT84	140	12.2
31OCT84	160	12.9
31OCT84	180	13.3
31OCT84	200	14.4
31OCT84	220	14.7
31OCT84	240	15.3
31OCT84	260	16.0
31OCT84	280	13.8
31OCT84	300	5.4
31OCT84	320	4.1
31OCT84	340	12.2
31OCT84	360	18.4
30NOV84	40	13.1
30NOV84	60	12.6
30NOV84	100	12.6
30NOV84	140	12.3
30NOV84	160	12.8
30NOV84	180	13.6
30NOV84	200	14.3
30NOV84	220	14.1
30NOV84	240	15.1
30NOV84	260	15.4
30NOV84	280	10.6
30NOV84	300	4.0
30NOV84	320	4.2
30NOV84	340	16.3
30NOV84	360	10.2
13DEC84	40	12.3
13DEC84	100	12.2
13DEC84	140	12.1
13DEC84	160	11.9
13DEC84	180	12.3
13DEC84	200	12.9
13DEC84	220	13.4
13DEC84	240	13.7
13DEC84	260	14.3
13DEC84	280	14.4
13DEC84	300	11.0
13DEC84	320	4.3

13DEC84	340	3.5
13DEC84	360	14.9
20DEC84	40	12.2
20DEC84	60	11.9
20DEC84	100	11.8
20DEC84	140	11.9
20DEC84	160	12.2
20DEC84	180	12.8
20DEC84	200	13.5
20DEC84	220	13.6
20DEC84	240	14.2
20DEC84	260	14.6
20DEC84	280	9.9
20DEC84	300	4.0
20DEC84	320	3.8
20DEC84	340	15.0
20DEC84	360	17.2
07JAN85	40	12.1
07JAN85	60	11.9
07JAN85	100	11.8
07JAN85	140	11.7
07JAN85	160	12.5
07JAN85	180	13.4
07JAN85	200	14.2
07JAN85	220	15.3
07JAN85	240	16.7
07JAN85	260	17.6
07JAN85	280	12.3
07JAN85	300	4.1
07JAN85	320	3.9
07JAN85	340	15.0
07JAN85	360	19.1
11FEB85	40	13.3
11FEB85	60	13.8
11FEB85	100	14.6
11FEB85	140	13.2
11FEB85	160	13.9
11FEB85	180	14.5
11FEB85	200	15.5
11FEB85	220	16.4
11FEB85	240	17.4
11FEB85	260	18.7
11FEB85	280	15.0
11FEB85	300	5.1
11FEB85	320	4.6
11FEB85	340	22.1
11FEB85	360	29.6
15MAR85	40	14.4
15MAR85	60	14.3
15MAR85	100	14.2
15MAR85	140	14.6
15MAR85	160	15.2
15MAR85	180	16.3
15MAR85	200	17.7

15MAR85	220	19.5
15MAR85	240	21.8
15MAR85	260	23.6
15MAR85	280	17.4
15MAR85	300	5.1
15MAR85	320	5.1
15MAR85	340	20.9
15MAR85	360	13.9
20MAR85	40	14.5
20MAR85	60	14.8
20MAR85	100	14.8
20MAR85	140	15.2
20MAR85	160	16.8
20MAR85	180	18.5
20MAR85	200	19.6
20MAR85	220	21.0
20MAR85	240	22.5
20MAR85	260	24.1
20MAR85	280	17.8
20MAR85	300	5.1
20MAR85	320	5.6
20MAR85	340	22.4
20MAR85	360	12.2
27MAR85	40	15.2
27MAR85	60	15.6
27MAR85	100	16.1
27MAR85	140	16.6
27MAR85	160	17.3
27MAR85	180	18.1
27MAR85	200	19.2
27MAR85	220	20.3
27MAR85	240	21.1
27MAR85	260	22.1
27MAR85	280	15.3
27MAR85	300	4.8
27MAR85	320	4.6
27MAR85	340	20.4
27MAR85	360	10.5
02APR85	40	17.6
02APR85	60	17.6
02APR85	100	17.5
02APR85	140	18.1
02APR85	160	18.3
02APR85	180	19.2
02APR85	200	20.2
02APR85	220	21.8
02APR85	240	23.3
02APR85	260	23.9
02APR85	280	16.7
02APR85	300	4.9
02APR85	320	5.6
02APR85	340	22.6
02APR85	360	11.6
25APR85	40	17.3

25APR85	60	16.7
25APR85	100	16.0
25APR85	140	15.9
25APR85	160	16.4
25APR85	180	16.9
25APR85	200	17.8
25APR85	220	18.4
25APR85	240	19.2
25APR85	260	19.4
25APR85	280	15.1
25APR85	300	4.7
25APR85	320	4.3
25APR85	340	15.4
25APR85	360	8.4
16MAY85	40	17.0
16MAY85	60	16.6
16MAY85	100	16.4
16MAY85	140	15.9
16MAY85	160	16.4
16MAY85	180	16.9
16MAY85	200	17.7
16MAY85	220	18.3
16MAY85	240	19.0
16MAY85	260	19.4
16MAY85	280	14.2
16MAY85	300	4.5
16MAY85	320	4.3
16MAY85	340	14.7
16MAY85	360	6.7
21MAY85	40	17.0
21MAY85	60	16.3
21MAY85	100	15.9
21MAY85	140	15.8
21MAY85	160	16.0
21MAY85	180	16.8
21MAY85	200	17.4
21MAY85	220	18.1
21MAY85	240	19.1
21MAY85	260	19.4
21MAY85	280	14.1
21MAY85	300	4.5
21MAY85	320	4.5
21MAY85	340	16.6
21MAY85	360	8.5
28MAY85	40	16.5
28MAY85	60	16.0
28MAY85	100	16.1
28MAY85	140	15.7
28MAY85	160	16.0
28MAY85	180	16.4
28MAY85	200	17.3
28MAY85	220	17.8
28MAY85	240	18.7
28MAY85	260	18.5

28MAY85	280	13.7
28MAY85	300	4.6
28MAY85	320	4.3
28MAY85	340	15.8
28MAY85	360	6.4
11JUN85	40	16.5
11JUN85	60	15.7
11JUN85	100	15.6
11JUN85	140	15.3
11JUN85	160	15.6
11JUN85	180	16.0
11JUN85	200	16.3
11JUN85	220	17.1
11JUN85	240	17.9
11JUN85	260	18.0
11JUN85	280	11.7
11JUN85	300	4.4
11JUN85	320	4.1
11JUN85	340	8.1
11JUN85	360	3.1
03JUL85	40	16.2
08JUL85	40	15.9
08JUL85	60	15.5
08JUL85	100	15.1
08JUL85	140	14.9
08JUL85	160	15.1
08JUL85	180	15.3
08JUL85	200	16.5
08JUL85	220	16.5
08JUL85	240	17.0
08JUL85	260	17.1
08JUL85	280	12.2
08JUL85	300	4.2
08JUL85	320	3.4
08JUL85	340	7.6
08JUL85	360	2.8
16JUL85	40	15.3
16JUL85	60	14.7
16JUL85	100	14.6
16JUL85	140	14.2
16JUL85	160	14.3
16JUL85	180	14.5
16JUL85	200	15.5
16JUL85	220	15.6
16JUL85	240	15.8
16JUL85	260	16.6
16JUL85	280	14.5
16JUL85	300	5.2
16JUL85	320	3.6
16JUL85	340	8.1
16JUL85	360	11.5
01AUG85	40	15.4
01AUG85	60	14.9
01AUG85	100	14.4

01AUG85	140	14.1
01AUG85	160	14.5
01AUG85	180	14.8
01AUG85	200	15.1
01AUG85	220	15.5
01AUG85	240	15.6
01AUG85	260	16.1
01AUG85	280	11.8
01AUG85	300	4.3
01AUG85	320	3.5
01AUG85	340	7.9
01AUG85	360	3.9
12AUG85	40	15.4
12AUG85	60	14.9
12AUG85	100	14.7
12AUG85	140	14.0
12AUG85	160	14.4
12AUG85	180	14.8
12AUG85	200	15.6
12AUG85	220	16.3
12AUG85	240	17.3
12AUG85	260	18.0
12AUG85	280	17.5
12AUG85	300	5.6
12AUG85	320	3.7
12AUG85	340	10.1
12AUG85	360	17.3

----- Subsidence Plot Tube Number - 403-----

Sampling Date	Depth From Surface (cm)	Volumetric Moisture Content (%)
07JAN84	40	14.4
07JAN84	60	13.8
07JAN84	100	13.5
07JAN84	140	13.8
07JAN84	160	14.2
07JAN84	180	15.1
07JAN84	200	16.4
07JAN84	220	17.3
07JAN84	240	17.3
07JAN84	260	17.6
07JAN84	280	18.2
07JAN84	300	6.3
07JAN84	320	4.0
07JAN84	340	10.3
07JAN84	360	14.7
23AUG84	40	12.8
23AUG84	60	14.7
23AUG84	100	15.6
23AUG84	140	16.0

23AUG84	160	15.9
23AUG84	180	16.6
23AUG84	200	17.6
23AUG84	220	18.7
23AUG84	240	18.6
23AUG84	260	6.6
23AUG84	280	4.5
23AUG84	300	4.6
23AUG84	320	5.1
23AUG84	340	9.9
23AUG84	360	9.4
27SEP84	40	14.3
27SEP84	60	14.5
27SEP84	100	14.4
27SEP84	140	14.9
27SEP84	160	15.1
27SEP84	180	15.2
27SEP84	200	15.8
27SEP84	220	16.4
27SEP84	240	15.9
27SEP84	260	16.1
27SEP84	280	17.1
27SEP84	300	6.4
27SEP84	320	4.2
27SEP84	340	8.2
27SEP84	360	9.0
31OCT84	40	15.2
31OCT84	60	15.3
31OCT84	100	14.7
31OCT84	140	15.1
31OCT84	160	15.3
31OCT84	180	15.4
31OCT84	200	15.8
31OCT84	220	16.9
31OCT84	240	16.9
31OCT84	260	17.6
31OCT84	280	18.6
31OCT84	300	8.7
31OCT84	320	4.2
31OCT84	340	7.2
31OCT84	360	16.6
30NOV84	40	15.4
30NOV84	60	14.8
30NOV84	100	14.6
30NOV84	140	14.8
30NOV84	160	15.2
30NOV84	180	15.4
30NOV84	200	15.9
30NOV84	220	16.4
30NOV84	240	15.0
30NOV84	260	15.8
30NOV84	280	16.3
30NOV84	300	5.8
30NOV84	320	4.0

30NOV84	340	9.5
30NOV84	360	8.4
13DEC84	40	14.7
13DEC84	60	14.0
13DEC84	100	13.8
13DEC84	140	14.0
13DEC84	160	14.2
13DEC84	180	14.2
13DEC84	200	14.8
13DEC84	220	15.3
13DEC84	240	14.6
13DEC84	260	14.8
13DEC84	280	15.4
13DEC84	300	5.6
13DEC84	340	3.6
13DEC84	360	8.3
20DEC84	40	14.6
20DEC84	60	14.0
20DEC84	100	13.6
20DEC84	140	14.2
20DEC84	160	14.4
20DEC84	180	14.2
20DEC84	200	14.6
20DEC84	220	15.1
20DEC84	240	14.6
20DEC84	260	14.7
20DEC84	280	15.3
20DEC84	300	5.8
20DEC84	320	3.7
20DEC84	340	9.0
20DEC84	360	18.6
07JAN85	40	14.4
07JAN85	60	13.8
07JAN85	100	13.5
07JAN85	140	13.8
07JAN85	160	14.2
07JAN85	180	15.1
07JAN85	200	16.4
07JAN85	220	17.3
07JAN85	240	17.3
07JAN85	260	17.6
07JAN85	280	18.2
07JAN85	300	6.3
07JAN85	320	4.0
07JAN85	340	10.3
07JAN85	360	14.7
11FEB85	40	16.3
11FEB85	60	15.7
11FEB85	100	14.9
11FEB85	140	15.4
11FEB85	160	16.0
11FEB85	180	16.3
11FEB85	200	16.9
11FEB85	220	18.5

11FEB85	240	17.4
11FEB85	260	17.4
11FEB85	280	17.9
11FEB85	300	6.9
11FEB85	320	4.3
11FEB85	340	10.8
11FEB85	360	27.6
15MAR85	40	17.1
15MAR85	60	17.0
15MAR85	100	17.2
15MAR85	140	18.0
15MAR85	160	19.1
15MAR85	180	19.6
15MAR85	200	20.7
15MAR85	220	21.7
15MAR85	240	21.6
15MAR85	260	22.3
15MAR85	280	23.0
15MAR85	300	7.3
15MAR85	320	4.7
15MAR85	340	12.4
15MAR85	360	14.4
20MAR85	40	17.7
20MAR85	60	17.7
20MAR85	100	18.4
20MAR85	140	19.6
20MAR85	160	21.1
20MAR85	180	21.7
20MAR85	200	22.5
20MAR85	220	22.8
20MAR85	240	22.1
20MAR85	260	22.0
20MAR85	280	24.2
20MAR85	300	7.2
20MAR85	320	5.0
20MAR85	340	13.8
20MAR85	360	11.4
27MAR85	40	19.5
27MAR85	60	19.5
27MAR85	100	19.9
27MAR85	140	20.2
27MAR85	160	21.0
27MAR85	180	21.6
27MAR85	200	21.6
27MAR85	220	21.5
27MAR85	240	20.2
27MAR85	260	20.2
27MAR85	280	20.2
27MAR85	300	6.8
27MAR85	320	4.6
27MAR85	340	10.6
27MAR85	360	9.7
02APR85	40	21.4
02APR85	60	20.7

02APR85	100	20.3
02APR85	140	21.2
02APR85	160	21.3
02APR85	180	22.3
02APR85	200	22.4
02APR85	220	22.6
02APR85	240	21.5
02APR85	260	21.7
02APR85	280	23.1
02APR85	300	7.4
02APR85	320	5.4
02APR85	340	13.7
02APR85	360	11.7
25APR85	40	20.0
25APR85	60	18.2
25APR85	100	18.1
25APR85	140	18.4
25APR85	160	18.6
25APR85	180	18.6
25APR85	200	18.9
25APR85	220	18.8
25APR85	240	17.4
25APR85	260	17.3
25APR85	280	17.5
25APR85	300	5.9
25APR85	320	4.1
25APR85	340	7.5
25APR85	360	5.6
16MAY85	40	19.2
16MAY85	60	18.3
16MAY85	100	18.3
16MAY85	140	18.4
16MAY85	160	18.9
16MAY85	180	19.0
16MAY85	200	19.0
16MAY85	220	19.1
16MAY85	240	17.8
16MAY85	260	17.6
16MAY85	280	18.0
16MAY85	300	6.4
16MAY85	320	3.8
16MAY85	340	7.2
16MAY85	360	4.0
21MAY85	40	19.6
21MAY85	60	18.5
21MAY85	100	18.2
21MAY85	140	18.3
21MAY85	160	18.7
21MAY85	180	18.6
21MAY85	200	19.1
21MAY85	220	19.0
21MAY85	240	17.8
21MAY85	260	17.7
21MAY85	280	18.3

21MAY85	300	6.5
21MAY85	320	4.3
21MAY85	340	8.2
21MAY85	360	6.8
28MAY85	40	19.4
28MAY85	60	15.5
28MAY85	100	18.1
28MAY85	140	18.1
28MAY85	160	18.7
28MAY85	180	18.4
28MAY85	200	19.0
28MAY85	220	18.5
28MAY85	240	17.6
28MAY85	260	17.0
28MAY85	280	17.9
28MAY85	300	6.3
28MAY85	320	4.1
28MAY85	340	7.8
28MAY85	360	5.4
11JUN85	40	19.0
11JUN85	60	18.0
11JUN85	100	17.5
11JUN85	140	17.9
11JUN85	160	18.0
11JUN85	180	18.3
11JUN85	200	18.0
11JUN85	220	18.2
11JUN85	240	16.7
11JUN85	260	16.3
11JUN85	280	16.9
11JUN85	300	6.2
11JUN85	320	3.7
11JUN85	340	5.5
11JUN85	360	3.5
03JUL85	40	15.2
08JUL85	40	18.8
08JUL85	60	17.9
08JUL85	100	17.3
08JUL85	140	17.0
08JUL85	160	17.6
08JUL85	180	17.3
08JUL85	200	17.4
08JUL85	220	17.4
08JUL85	240	16.1
08JUL85	260	15.9
08JUL85	280	15.7
08JUL85	300	6.0
08JUL85	320	3.8
08JUL85	340	5.2
08JUL85	360	2.9
16JUL85	40	18.2
16JUL85	60	17.2
16JUL85	100	16.3
16JUL85	140	16.8

16JUL85	160	16.6
16JUL85	180	16.9
16JUL85	200	16.5
16JUL85	220	16.9
16JUL85	240	15.8
16JUL85	260	15.1
16JUL85	280	16.1
16JUL85	300	7.2
16JUL85	320	3.8
16JUL85	340	6.0
16JUL85	360	11.6
01AUG85	40	18.2
01AUG85	60	16.9
01AUG85	100	16.6
01AUG85	140	16.1
01AUG85	160	16.6
01AUG85	180	16.5
01AUG85	200	16.8
01AUG85	220	16.6
01AUG85	240	15.2
01AUG85	260	14.2
01AUG85	280	14.6
01AUG85	300	5.8
01AUG85	320	3.5
01AUG85	340	5.3
01AUG85	360	3.6
12AUG85	40	18.0
12AUG85	60	17.3
12AUG85	100	16.5
12AUG85	140	16.7
12AUG85	160	16.6
12AUG85	180	16.7
12AUG85	200	16.2
12AUG85	220	16.8
12AUG85	240	15.8
12AUG85	260	15.0
12AUG85	280	15.8
12AUG85	300	8.1
12AUG85	320	4.0
12AUG85	340	6.3
12AUG85	360	15.1

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