SUBSIDENCE AND SETTLEMENT AND THEIR EFFECT ON SHALLOW LAND BURIAL

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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. Examples of expected settlement and subsidence are calculated bases 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. Finally, we briefly comment on our current field experiment, which studies the influence of subsidence on layered systems in general and on biobarriers in particular.

INTRODUCTION

Evidence of Occurrence

In a staurated 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.

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 adjacent soils, have been published on the subject.[1-5]. Consolidation and shear strength are discussed below. Uneven settlement or differential settlement is by far more damaging to a pit overburden than is total settlement. It 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 I found the terms are often used interchangeably. The nonhomogeneity of the buried waste and containers is the major cause of 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.

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:

- Consolidation.
- Lateral and upward expulsion of cohesionless or saturated soil masses.
- Cave-ins resulting from the following:
  - Unbraced excavations such as shallow land burial pits either before or after backfilling. This is caused by exceeding the shear strength of the slope.
  - Rotting or degradation of the waste products serving as braces and support for the overburden or backfill.
  - Slumping of the overburden, which is caused by movement of soil particles into existing interstices between waste containers.
  - Decreased soil shear strength through wetting.
  - Large-scale dewatering.
- Inadequate soil compaction.

GEOTECHNICAL TESTING

Densification

Several interdependent mechanisms contribute to the densification of granular soils:

- Compression of air and water in the voids.

- Squeezing of air and water out of voids.
- Permanent deformation caused by crushing of particles.
- Elastic deformation caused by bending of particles.
- Rearrangement of particles caused by sliding and rolling of particles relative to one another.

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 and on the relative density of the granular soil in question.

Consolidation

Loading a soil with a man-made 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 cause settlements.

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

$$\Delta H = \frac{PH}{EA}, \qquad (1)$$

with

E = elasticity modulus in Pa,
P = load in N,
H = thickness of soil layer in m,
A = area of soil under stress.

The elasticity modulus is determined by the slope of the initial stress/strain curves.

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. 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 $m_v = d\varepsilon/d\sigma$ with σ = stress and $\varepsilon = \Delta H/H$ 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

$$a_v = de/d\sigma = m_v(1+e), \qquad (2)$$

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 index, C_c, is the slope of the straight line, where $\Delta e = -C_c \log\sigma/\sigma_o$.

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

$$C_v = \frac{k}{\lambda_w m_v} \qquad (3)$$

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

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 driven by a particular mass dropped from a predetermined height over a certain distance. Peck et al.[6] relate allowable net bearing pressure, p (in kPa), to settlement, ΔH (in mm), as:

$$p = 0.41\, N\, \Delta H. \qquad (4)$$

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 since it has been shown[6] that the standard penetration blow count increased with increasing effective overburden pressure. The correction factor

$$C_F = 0.77 \log \frac{1915}{\sigma} \qquad (5)$$

with σ in kPa.

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

weight of set soil: 1.6×1000 kgm^{-3} $\times$ 9.81 ms^{-2} = 15.7 kNm^{-3}.

Overburden pressure: $15.7\ kNm^{-3} \times 8m = 126$ 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

$0.41 \times 27 \times 100 = 1107$ kPa = 1.1 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. The best estimates of settlement can be obtained by using, according to Lambe and Whitman:[7]

- Elastic theories to estimate stresses.
- Obtaining strains or elasticity moduli.
- Relying upon experience to compensate for sample disturbance.

Soil Failures Settlement

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 $13.7\ kNm^{-3}$. We are asked to predict the settlement of an underlying layer of 10m of that sandy silt if the same material is used as backfill:

At mid-depth in the sand, the stress,

$\sigma = 5m \times 13.7\ kNm^{-3} = 68.6$ kPa.

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

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

Settlement
$$\Delta H = \frac{\Sigma H_o}{1 + e_o} \Delta e \qquad (6)$$

or

$$\frac{10\,m}{1 + 0.83} (0.06546) = 0.36\,m .$$

(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, since $\frac{\Delta e}{1+e} = m_v \sigma$,

$$\Delta H = \Sigma H_o\, m_v\, \Delta\sigma \qquad (7)$$

or

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

since $a_v = (1+e)m_v$,

$$\Delta H = \frac{\Sigma H_o}{1 + e_o} a_v \Delta\sigma \qquad (8)$$

or

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

since $\Delta e = - c_c \log \frac{\sigma}{\sigma_o}$,

$$\Delta H = \frac{\Sigma H_o}{1 + e_o} C_c \log \frac{\sigma_f}{\sigma} \qquad (9)$$

or

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

The four methods yield $\bar{x} = 0.31$m and $s = 0.04$m. This indicates a remarkable agreement if one considers that the average volume compressibility m_v and the average coefficient of compressibility a_v were calculated from the $\bar{m}_v$ and $\bar{a}_v$ values at $\tau = 120$ kPa and 250 kPa found in "Geotechnical Aspects of Hackroy Sandy Loam and Crushed Tuff.[8] The stress was computed for mid-depth (stress increases directly proportional to depth).

A refined method (Holtz and Kovacs)[9] 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 or $\Delta H_1 + \Delta H_2 + \Delta H_3 + \ldots$No shortcut should be made by averaging estimated individual stratum settlement because each stratum is likely to possess a very proper and different coefficient of consolidation; therefore, each stratum must be analyzed individually.

Sowers[10] 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 a waste disposal site, could be quite deep. Deeper that 2B, there is very little settlement from any surface load because E, the elasticity mudulus, increases with depth and confinement, whereas the effects of any surface load decrease rapidly with depth.

Compression of sandy soils is rarely observed because consolidation is immediate and occurs 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 and subsequent settlement and the 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 of 10m of such a saturated mix provided the same material as in the previous case was used as backfill (8m of backfill having a unit weight of 13.7 kNm^{-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: $4.16 \times 10^{-7} m^2 s^{-1}$;

Unit weight: 16 kNm^{-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 −5 m: $\sigma_{(-5)} = 16\,kN\,m^{-3} \times 5\,m = 80\,kPa$,

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

2.) Stress increase due to backfill:

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

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:

$$\Delta H = \frac{\Sigma H_o}{1+e_o} C_c \log \frac{\sigma'_{(-5)}}{\sigma_{(-5)}} =$$

$$\frac{10\,m}{1+0.757} \times 0.145 \log \frac{189.6\,kPa}{80\,kPa} = 0.31\,m$$

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 I), can be used. Suppose we divided each layer in thicknesses of 1m each. The mid-depths are then, respectively, at d (in meters) with corresponding values of $\sigma, \sigma', \sigma'/\sigma, \log \sigma'/\sigma$, and

$$\frac{\Sigma H_o}{1+e_o} C_c = 0.0825\,(\text{constant}) \,. \tag{10}$$

In this case, $\Sigma\Delta H = 0.40m$, which is a more accurate result. 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%. It is generally agreed that consolidation settlements can only be predicted within a range of 20%.[9]

TABLE I

Settlement Computations

d(m)[a]	σ(kPa)[b]	σ′(kPa)[c]	σ′/σ	log σ′/σ	ΔH(m)[d]
0.5	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.

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 10m. The coefficient of consolidation approximates $1m^2$/month. Based on the above, we can construct Table II.

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.

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, by pumping of water or oil and causing some kind of passive consolidation. This, in turn, causes the ground surface to sink selectively.

TABLE II

Time Rate of Settlement

U	T_v	ΔH(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 burials can have the same effect.

Bracing of any excavation is required to prevent the phenomenon known as "lost ground,"[10] 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.

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, is 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 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 on its geotechnical performance. This may be intended to increase the elasticity modulus, which in turn may imply an increased strength or decreased compressibility or both. Soil improvement may be temporary in intent, or permanent. Lambe and Whitman[7] 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.

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 vs 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 increase to a peak and subsequently decrease). We see in Fig. 1 that a maximum dry density of 1.83 (17.85 kNm^{-3} unit weight) is reached at an optimum moisture content of 13% for a bentonite/sandy silt ratio of 0.02.

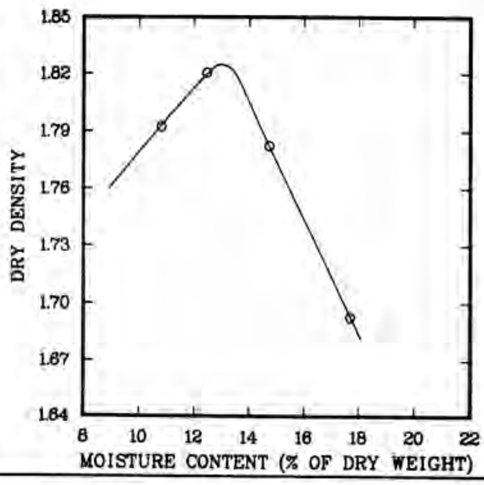

Fig. 1. Compaction test.

Figure 2 shows clearly that by decreasing the compactive effort the maximum dry density lowers in value and the optimum water content increases.[11] 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 only shows that a lower dry unit weight corresponds to a higher void ratio or porosity.

The decrease in void ratio can also have drastic consequences on the hydraulic conductivity, as depicted in Fig. 3 where void ratio is plotted against hydraulic conductivity for different bentonite ratios. Lutton et al.[11] show the same effect taking place on different materials (Fig. 4). They also show the effect of void ratio on the angle of internal friction where it is clearly shown that, for any particular soil, a decrease in void ratio inevitably leads to soil stabilization because of a higher angle of internal friction (Fig. 5). It should be kept in mind that both a soil strength increase and a reduced permeability resulting from one form or another or compaction affect the integrity of a waste pit cover positively.

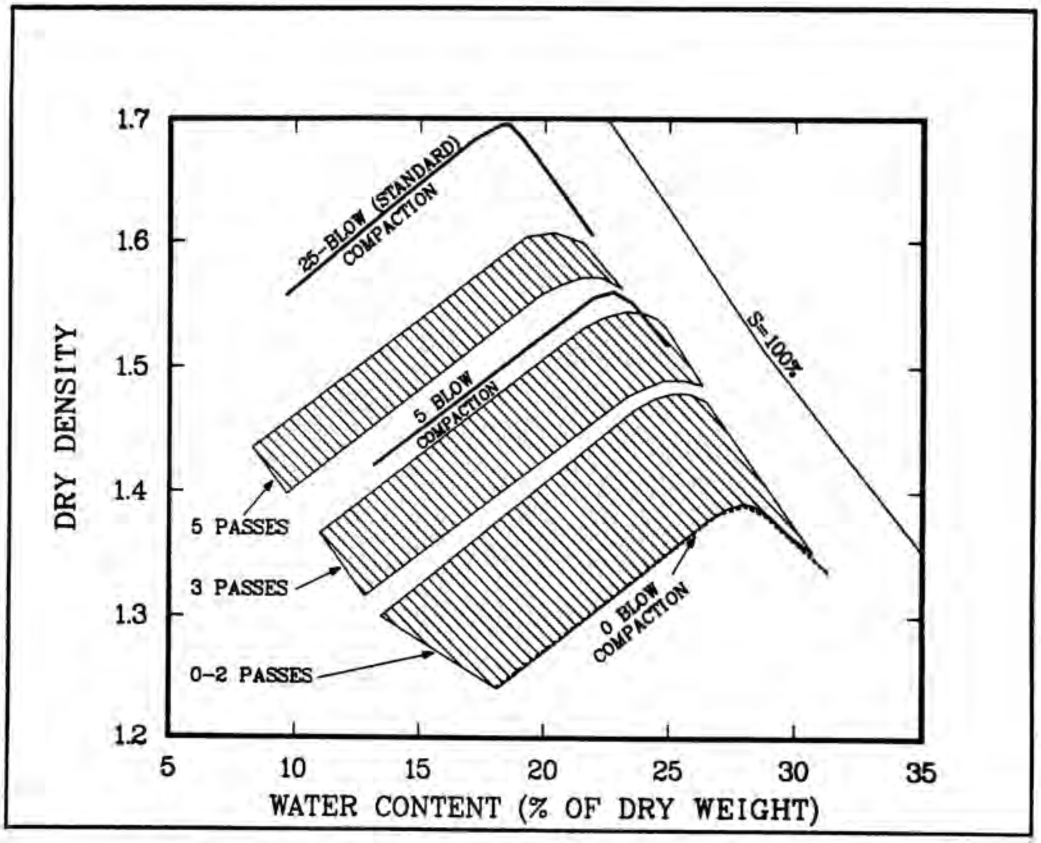

Fig. 2. Dry density as a function of water content.

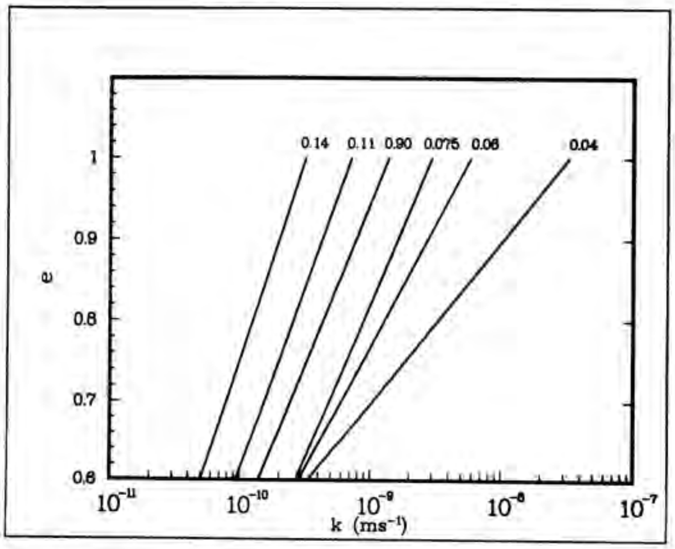

Fig. 3. Hydraulic conductivity as a function of void ratios for varying clay contents.

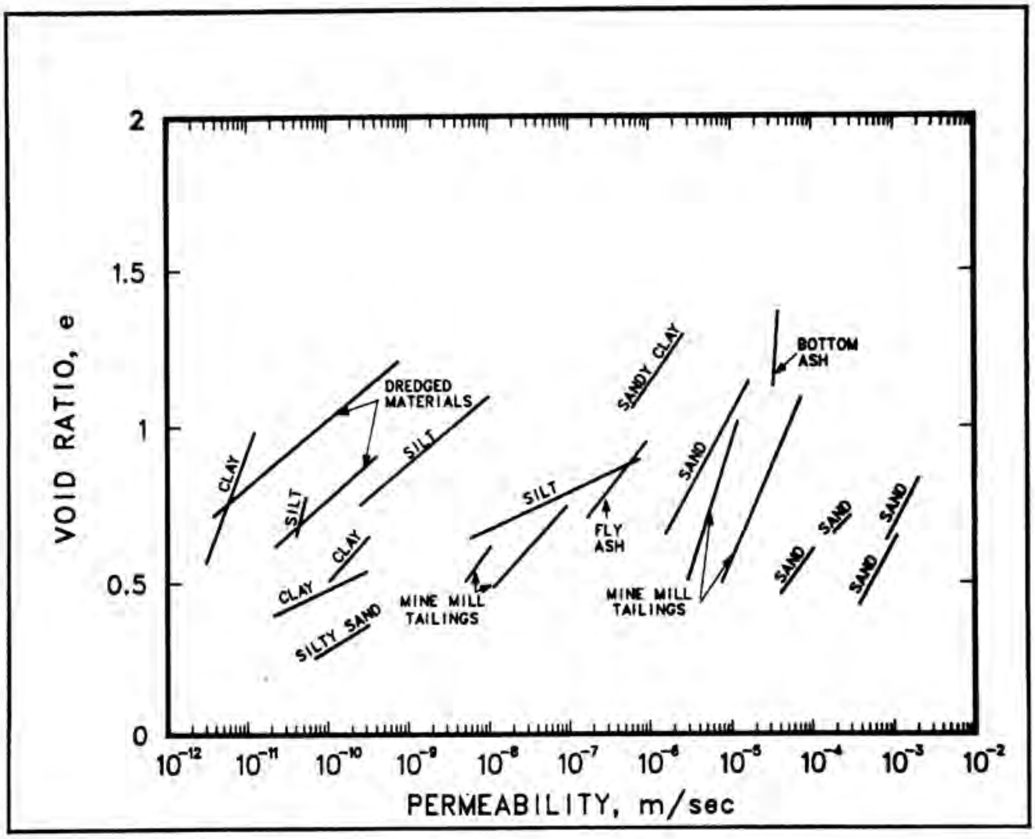

Fig. 4. Permeability of materials as affected by void ratio.

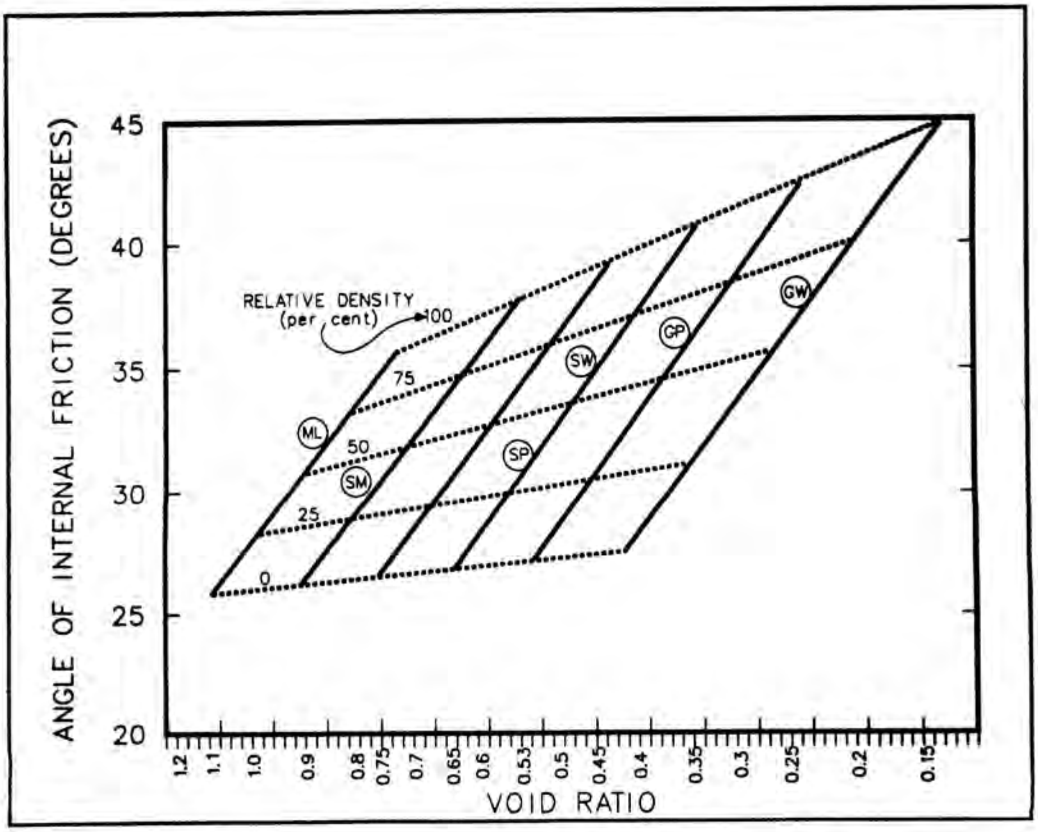

Fig. 5. Relation of effective angle of internal friction to void ratio for various soil types.

Although soils compacted over waste pits are generally relatively soft, one should strive for, on a granular soil-like solid waste, 90% of maximum dry density obtained by the 25-blow standard compaction test. Figure 6 shows how compaction curves vary with various soil types.[11] S. Phillips[12] shows that, by mixing Styrofoam in a ratio of 1:1 to silty sand, the coefficient of compressibility, a_v, which varied from 1.88 X $10^{-7}Pa^{-1}$ to 1.04 X $10^{-7}Pa^{-1}$ for corresponding stress intervals of 0 to 239 kPa and 239 to 575 kPa for silty sand, adopted values varying from 1.52 X 10^{-6} Pa^{-1} to 2.17 X 10^{-6} Pa^{-1} for the mix. Thus, he concludes 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 changed 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 (γd) and submerged tuff (γs) is equal to:

$$\gamma d/\gamma s = \frac{(1-n)\,G\gamma_w}{(1-n)(G-1)\gamma_w} \qquad (11)$$

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 means that the unit weight of crushed tuff is approximately 1.6 times higher when dry than when 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 2mm 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 cause of the literal "disappearance" of the city. The volcanic ash, at the origin of the bentonitic clay,

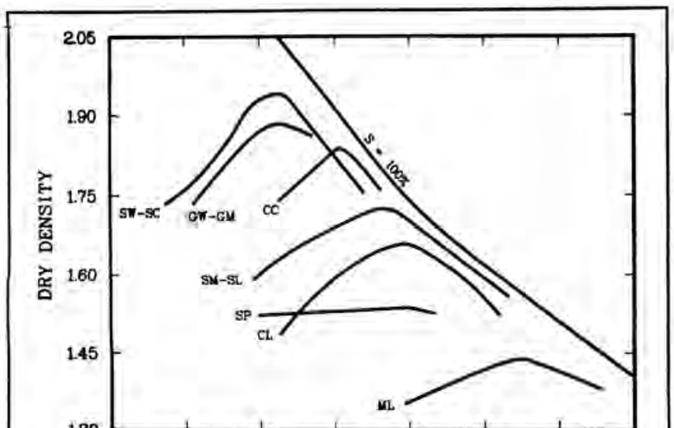

Fig. 6. Compaction curves for various soil types.

has a unit weight averaging only 6 kNm^{-3}, and consequently, is very compressible when loaded (as by dewatering).

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

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

FIELD SUBSIDENCE EXPERIMENT

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 2.75m in diameter and 95% are less than 4.25m 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-m^2 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 to prevent clogging. The backfill is 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, 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.

Preliminary Results

The resistance to subsidence should be equal above all eight drawholes since 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. We were even told by the explosives experts that two of the explosive bolts might not have detonated. For some time it looked as if subsidence would occur by accident (as it eventually does in a completely natural environment) or else, some method had to be found to induce or enhance subsidence without using disruptive mechanical means, which would leave a permanent imprint of "artificial" intervention.

does in a completely natural environment) or else, some method had 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.[8] 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 to be 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.

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 is far from resembling an inverse cone with regular slope. Instead, it has, in most cases, a vertical wall where the cohesive materials are located (Hackroy series soil), and extremely irregular angles where the diameter of the unstable moving material is not small compared with the height of the slope (gravel and cobble in our case). The ratio of the diameter of the unstable moving material to the total slope has to be small to satisfy the demand for identification of 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 ratio diameter/length of the slope is too high in the case of gravel and cobble and the compression is too high in the crushed tuff for their slope angle to be representative of the angle of repose. Cohesion prevents the Hackroy series soil from adopting an angle that is indicative of what the angle of repose might be.

The total volume of the subsided cavities seems to relate pretty well as 1:2.5:58.5 or roughly the relationship existing in the size of the respective drawholes. 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:

- The stratification is horizontal (soil, biobarrier, tuff).
- The subsidence reached its final stage.
- The cavities are geometrically simple.

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

- The surface subsidence boundaries extend beyond the edges of the cavity.
- Concurrent with subsidence, horizontal displacements producing stresses occur. Those movements are larger than would be expected from the subsidence curvature.
- 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 only present 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. 7). Prediction of maximum subsidence is based on the fact that it is correlated to cavity thickness, or $S = at$, 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. Since this was not the case, we can assume that every cavity was 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 on 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[13] suggests

$$S = \frac{25\,m}{25+\sqrt{h}}\cos\alpha\ , \qquad (12)$$

where α = angle of dip, and h = depth of cavity.

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

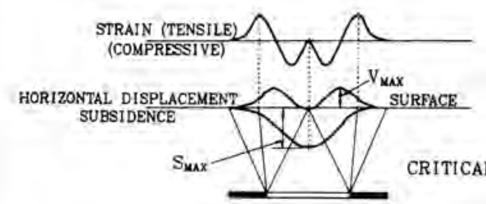

Fig. 7. Subsidence and surface movements.[14]

The National Coal Board, Mining Department[15] 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, W, or at depths, h, of less than 50-m. Since the W/h ratio is important for the use of these curves (named S/m curves), it would be interesting to see what would happen if W and h were multiplied by constant factors to see what S/m ratio (Table III) could be read on the curve used to compute subsidence based on the width and depth relationship. (In our case, W = 0.90-m and h = 3.1-m.)

If we take the averaged obtained S/m value (0.2) and extrapolate to a depth of 3.1m, then we find (Table IV) that for cavities of thickness, t, the predicted vs measured subsidence agree reasonably well.

As we can see, even though this method may seem unorthodox, only the deepest cavities show the highest deviations.

Biointrusion

Statistical analyses of data from the short-term, small-scale biointrusion studies conducted in lysimeters[16] 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 (7.5- 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:

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

TABLE III

Relationship of Subsidence to Width (W) and Depth (h)

W(m)	h(m)	W/h	S/m
90	310	0.29	0.18
135	465	0.29	0.22
180	620	0.29	0.20
225	775	0.29	0.19
270	930	0.29	0.22

TABLE IV

Predicted (S_p) vs Measured Subsidence (S_m)

t(m)	S_p(m)	S_m(m)
1.4	0.3	0.4
3.4	0.7	0.8
6.4	1.3	1.3
11.5	2.3	1.6

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

Evaluating the effectiveness of the soil/rock intrusion barrer 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 CsCI 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.

Results and Discussion

For reasons discussed previously, the surface subsidence craters never materialized upon opening of the covers over the subsurface void spaces. Consequently, none of the cesium concentrations that were measured in plants are indicative of the effects of subsidence on barrier integrity. However, those data do indicate short-term performance of the soil/rock cap design at intermediate scale.

Cesium concentrations in vegetation, averaged over the entire subsidence plot area, are presented in Table V. Note that the concentrations all averaged 1 ppm, indicating that the soil/rock cap design effectively limited root access to the cesium.

ACKNOWLEDGMENTS

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

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TABLE V

Average Cesium Concentrations in Vegetation from the Subsidence Experiment During the 1983 Growing Season

Date	Cesium Concentration (ppm, n = 8) Mean[a]	S.D.
July 11, 1983	0.14	0.26
August 15, 1983	0.91	0.53
September 13, 1983	0.08	0.07
October 4, 1983	0.50	0.31
November 8, 1983	0.50	0.60

[a]Cesium concentrations are averaged over the entire plot area because of a lack of subsidence treatment effects.