Compacted Earthfill
is a
3-phase Material

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Abstract

This article describes the physics involved at each stage around the hysteresis loop which encapsulates the drying-wetting cycle. As a 2-phase dry soil-structure (solids and air) progressively takes in water, thereby becoming a 3-phase system, it eventually becomes a water-saturated 2-phase system. The reciprocal withdrawal of water during the process of returning to the dry condition follows a different working path, thereby involving a hysteresis effect which highlights the work needed to return to the original state.

To allow a mathematical treatment of these physics, an idealized model aggregation of uniform-sized spheres, arranged both in their loosest and densest packing arrangements, is examined. Graphic representations of the work/energy hysteresis are shown plotted against normalized axes which are independent of particle size and moisture content.

Computer coding is provided for the permeability (hydraulic conductivity) associated with any state of these multi-phase aggregations.

Soil Physics laboratory-derived drying cycle plots are given for three sand size fractions made from a natural soil. A comparison between the behaviour of these real materials and the idealized spherical model are discussed, together with practical inferences which may be drawn from their gradations or particle shapes.

The unexpected post-construction behaviour of two highly significant engineering structures (Tarbela and Bennett Dams) caused serious concerns regarding their stability, and consequently, resulted in the expenditure of huge resources. Both the fears and the costs might perhaps have been lessened had the controlling engineers appreciated that they were merely witnessing the time-dependant aspects of a 3-phase material responding to increasing moisture content.

1. Introduction

It is beyond the reach of theoretical physics to anticipate the detailed behaviour of soils: the clear impossibility of fully defining the exact material idiosyncrasies of the grain aggregation simply has to be faced. But it is equally clear that any possibility of putting rational boundaries, and perhaps suggesting limits, or even defining the kinemathics of inter-particulate contacts could be useful. It is in this sense/spirit that the following ideas are put forward.

The approach adopted here is to simplify this intractable problem by modelling the sand- and silt-sized soils of primary importance here as if they were perfect spherical solids of uniform size, neatly arranged into well defined geometric arrays. It is easily argued that examining the implications of particle size and moisture content of partially saturated soils as if these were well ordered arrays of spheres can have little to do with, or tell us anything about, the real world of geotechnical engineering. And having spent my best years working on earthworks construction in the field, freely acknowledges this deficiency. But then, whatever else can we do to move above and beyond ground zero.

It is worth noting that there are excellent precedents for this simple approach to engineering research. The frontispiece of the inaugural issue of Geotechnique is a picture of Osborne Reynolds holding before him a container of steel balls all of the same size. Presumably this symbolized what he wanted to emphasise as a good avenue to future speculation. Somewhat later, Peter W. Rowe, his successor at the University of Manchester made much the same point to in his 1969 Rankin Lecture entitled "Osborne Reynolds and Dilatancy". This approach seems to have gone out of fashion probably because later generations supposed such eminent engineers and thinkers must have exhausted this means towards enlightenment.

Anyway, the plain fact of the matter is that in order to make any advance in applied physics (engineering) it is first necessary to come to know the main principles which govern the kinematics of the system under investigation. Then, using these rules as guidance, design/devise laboratory testing to tease out an understanding of the degree to which real materials depart from those idealizations of behaviour.

2. Particle Packing

The geotechnical term most commonly used to define the extent of particle packing density is void ratio (e). This is the ratio of void space divided by total volume, that is, voids plus solids. An equivalent term, porosity (η), is related such that $\eta = e/(1+e)$.

The two extreme packing arrangements for uniformly sized spheres are studied herein. The loosest possible packing has a void ratio where $e = (6/\pi) - 1$, or $\eta = 0.476$. It is called "cubic open" packing and is illustrated in Figure 1a where it may be seen to consist of square arrays set directly on top of each other. There are two entirely different ways of making the densest packing, both having $e = 6/(\pi\sqrt{2}) - 1$, or $\eta = 0.260$. The "cubic close" is where a square array sits in the troughs of the layer beneath, as is

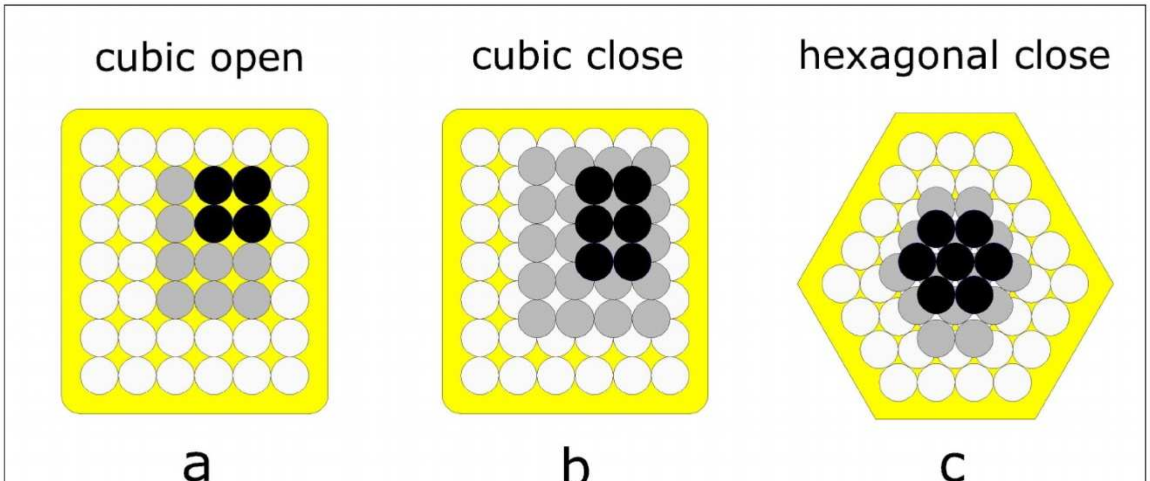

Figure 1

shown in Figure 1b. The "hexagonal close" depicted in Figure 1c, which is the dense packing adopted herein, is formed of triangular arrays nesting in the troughs beneath. Particles in the loosest arrangement touch 6 neighbouring spheres, the cubic close 8, and the hexagonal close 12.

3. Meniscus Forces & Volumes

Having conceded the need to adopt uniformly sized spherical shapes in regular arrays as a first approximation to aggregations of soil grains we can now move towards determining the idealistic principles which govern earthfill behaviour.

Figure 2 shows the Loose (cubic open) and the Dense (hexagonal close) which are the two extreme packing densities considered hereinafter. In this sketch they are shown with water adhering/attached to them as menisci. This is one case in the range of the 3-phase system of solid, water and air. Here the menisci are shown at their maximum extent/volume where Θ is 45° for the Loose case and 30° for the Dense case. This is a threshold condition where any increase in moisture content will cause the menisci to collapse into a new configuration which will be discussed later.

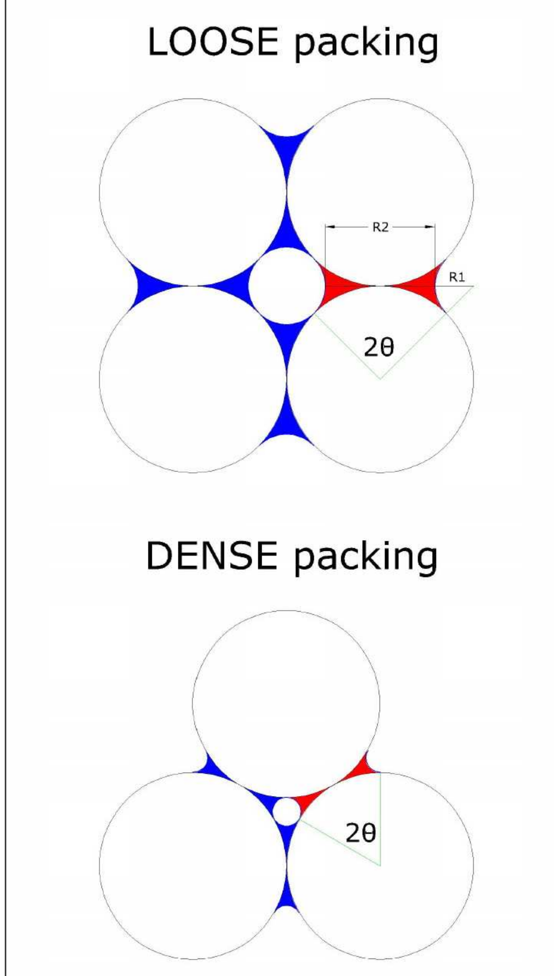

Figure 2

From the geotechnical point of view it is the surface tension force (T) which is of primary interest since it dictates the pressure difference between the liquid and vapour phases. The following set of equations allow calculation of the pressure deficiency (PD) between the water droplet with respect to the void air:

$$PD = T\,(1/R1 - 1/R2)$$
$$R1 = r\,(\sec 2\Theta - 1)$$
$$R2 = (r \tan 2\Theta - R1)$$

r = radius of the spherical particle, and

T = 0.0741 N/m at 10 °C.

The complementary relationship between the volume of one complete meniscus droplet (V) and particle size and extent of wetness is:

$$V = 8 \pi r^3 \sin^4 \Theta [1 - (\pi/2 - 2\Theta) \tan 2\Theta] / \cos^2 2\Theta.$$

These equations, together with Boyle's Law allow to be drawn the full drying-wetting hysteresis curves shown in what followings.

4. Drainage of Pore Water from Voids

One of the most important lessons we geotechnical engineers can, and indeed ought to, learn from Soil Physics is that once a saturated soil is allowed to drain it is not an easy thing to re-saturate the soil-structure; this is particularly difficult in finer grained soils. The goal of this technical essay is to make this reality clear and, furthermore to point out the consequence this apparently esoteric fact of nature can have on earthfill structures, especially water retaining embankments.

In Soil Physics this phenomenon is referred to as the "ink-bottle effect". This out of date allusion is/was intended to illustrating the essence of the situation. A very simple experiment makes the point: Find a small bottle with a choked neck, fill it with water and invert it over standing water. The water will remain inside the bottle and it will not drain out until some air is allowed to enter so as to release it. Now, to simulate a falling water table, lift the rim of the bottle above the outside water level and, of course, the bottle will empty. But, to simulate a rising water level, push the rim of the empty bottle underwater, and it will be seen not to admit the outside water. Obviously this is because the air cannot get out to let the water in. Kindergartenesque though this experiment appears it nevertheless illustrates the physical principle which prevails in the void spaces within an unsaturated particulate mass. It is easier to empty the voids than to refill them.

At a somewhat more refined level of experimentation, the results shown on Figure 3 are from my own laboratory work for my masters degree. The Soil Physics unit/measure of suction (sub-atmospheric pressure), used during those experiments was pF, where pF is defined as Log_{10} cm H_2O, and therefore, is used here too. Various soil gradations (sands, silt, and clay) were tested throughout their full drying range from water-saturation to air-saturation. Only the early, vapour condensation, stage of re-wetting was achieved.

The three sands shown here are fractions of the same natural soil. The particle sizes of the coarse specimen were from 2.0mm to 0.2mm, the fine material 0.2mm to 0.05mm, and the third specimen was a mixture of 40% coarse to 60% fine sand. The nature of the testing procedures ruled out compaction of the specimens. Nonetheless, the moisture contents of the coarse and fine specimens, starting out above 25%, implies looseness. Incidentally, the lower initial amount of water in the mixed specimen can be attributed to its better gradation.

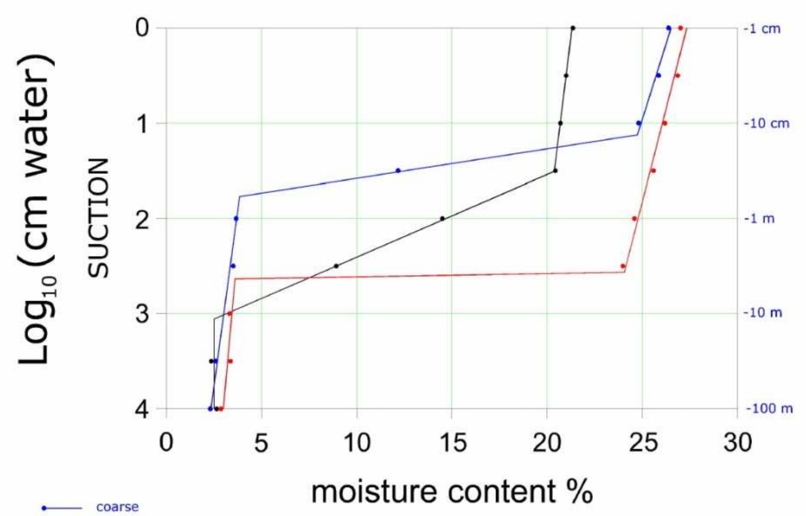

Figure 3

The test results from the coarse specimen show a gradual water loss from ambient pressure down to 1pF, and then between 1pF and 2pF, 73% of the water content is lost to pore drainage. This response to imposed suction indicates (as will be explained later) that 73% of the pores have an equivalent circular diameters varying between 0.30mm and 0.03mm. The fine specimen shows a gradual loss from 0pF to 2.5pF and then 75% of the water drains between 2.5pF and 3pF. The equivalent circular pore diameters corresponding to these suctions are 0.01mm and 0.003mm. The moisture retention characteristics of the mixture shows 88% of the water was lost between 1.5pF and 3.5pF. These suctions correspond to pore sizes of 0.05 mm and 0.0005 mm. This upper limit is less than the largest pores in the coarse sand, and the lower limit is only a third of the size of the smallest pores in the fine sand. This overall reduction in pore dimensions is caused by the finer grains particles filling some of the pore spaces formed between the larger grains.

5. Filling dry Voids with Water

Even back in my student days the full drying curve had been defined all the way from ambient pressure down to absolute zero. But that was not the case with the reciprocal/reverse process of rewetting the dried-out pores. I presume this was because in Soil Physics there was little of practical interest associated with this aspect of soil behaviour. Perhaps this deficit has been rectified in the meantime and I am unaware of it.

In any event it has been something I wanted to sort out for myself – to complete the cycle – to close the loop of drying and rewetting of the voids within the soil-structure: Basically, to see for myself if this phenomenon has/had any bearing on how we go about using Soil Mechanics. Just recently I have found time to do just that, and the purpose of this technical essay is to document what came out of that effort. With that aim in mind I now propose closed curves (e.g. Figures 4 & 5) which I believe represent the loci of a continuous relationship between moisture content and ambient water pressure.

So, using the above as a prelude, we can now move on to teasing out any geotechnical significance that may adhere to these hysteresis curves. But before doing so it is necessary to say something about the coordinate values against which the data points are plotted. We are quite familiar with the abscissa with its measures of either water content or percent saturation, but the ordinate scale needs some introduction/explaining.

A choice has been made to use a compound-logarithmic ordinate scale for the following reason: Because of the great spread/range of the pressure data of interest, a logarithmic scale is necessary to cover the full scope of activity. And then, to have adopted absolute zero as the basis of measurement would seem a foregone conclusion, thereby making any distinction between suction and gauge pressure unnecessary. But unfortunately that would have resulted in the most interesting and informative data appearing in the Log_{10} 3 region of the plot where detail would be virtually indecipherable. So, in order to provide data clarity in the vicinity of atmospheric pressure, while still accommodating very high values, two separate log scales were needed - a pressure scale set directly above a suction scale.

However, it must be acknowledged that this procedure involves a compromise with mathematical propriety: Since the unit weight of water is

$1gmf/cm^3$, the zero-ordinate ($Log_{10} = 0$) has the value 1cm H_2O pressure on the top side of the line, whereas the same line, on the suction side representing 1cm H_2O of negative pressure. Therefore, a line which should have no thickness, here turns out to be 2cm wide. Nevertheless, I decided to gloss over this mathematical nicety in favour of clarity of depiction.

6. Going around the Hysteresis Loop

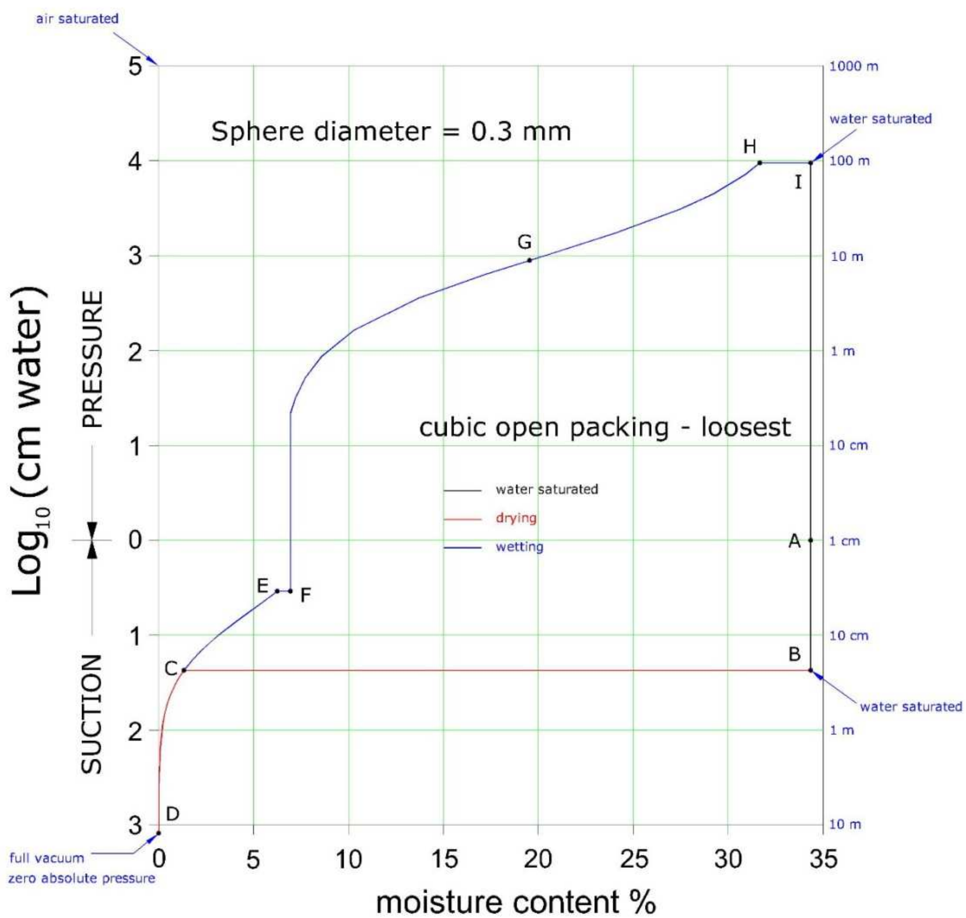

Figure 4

Figure 4 is a graphical representation of the full wetting-drying cycle for uniformly sized 0.3mm solid spheres arranged in their loosest packing arrangements. This specific particle size was chosen because it is of interest to me with regard to earthquake liquefaction. In the generality of 3-phase physics the void space between the solid particles can be occupied by either liquid or vapour, or some combination of both. In geotechnical engineering the liquid is water and the vapour is air.

Point **B** is reached from Point **A** by depressing the ambient water pressure by the amount necessary to create a suction which will permit an air bubble to enter the void spaces through the openings available between the solids. This, in a mass of uniformly sized spheres, results in the drainage of most of the pore water. So, the system then passes from 2-phase to 3-phased.

At Point **C** the loss of water comes to a halt, and cannot be further reduced unless there is a further depression in pore water potential. The reason for this impediment is that any remaining water is held within the menisci where particles touch, which incidentally, at this point are holding only slightly more than 20% of their water capacity. The continuity of the water phase has therefore been broken, and phase continuity now resides in the pore air alone.

To reach Point **D**, where pore energy is zero, further drying is necessary and this may be brought about by evaporation or transpiration. The outside ambient water level is irrelevant.

Similarly, Point **E** can only be reached from **D** or **C** by increased humidity and condensation of vapour into the menisci, thereby increasing the moisture content while lessening the suction level (increasing the pore water energy).

Point **E** is a state of extreme instability. This is because the menisci's ability to hold water are at their full capacity and these are on the point of brimming over as depicted in Figure 2: They cannot expand without interfering with adjacent menisci. I was stuck for a long time at this apparent impasse. Finally, however, it became apparent that the only physically plausible outcome under the conditions that pertained there and then was a catastrophic inversion of the air/water interfaces. An event where/when the menisci surfaces flip to a complete reversal of their previous role - from one of constraining the water against the particles, to one of containing the void air within a modified bubble.

This idea is entirely consistent with the understanding that the natural tendency of surface tension is to minimize it's area (reduce energy potential) and the fact that at this physical/theoretical crisis point the combined area of the menisci surfaces are more than necessary to enclose the existing void air in a bubble of modified shape. During the ensuing implosion there is an energy release without any possible response from the prevailing outside ambient energy of the system as a whole. And it is in order to rectify this nonconformity that insists on the next position along the wetting cycle.

Point **F** is calculated on the understanding that the excess energy released by the collapse is expended by the work done in compressing the pre-existing air volume to a new, smaller, volume. And since the new pressurized volume is contained within a bubble, which is a free-body of balanced tensile and compressive forces, there is no attendant change in the ambient energy level. Consequently, **F** can only exists at a slightly higher pore water content appropriately removed from **E**.

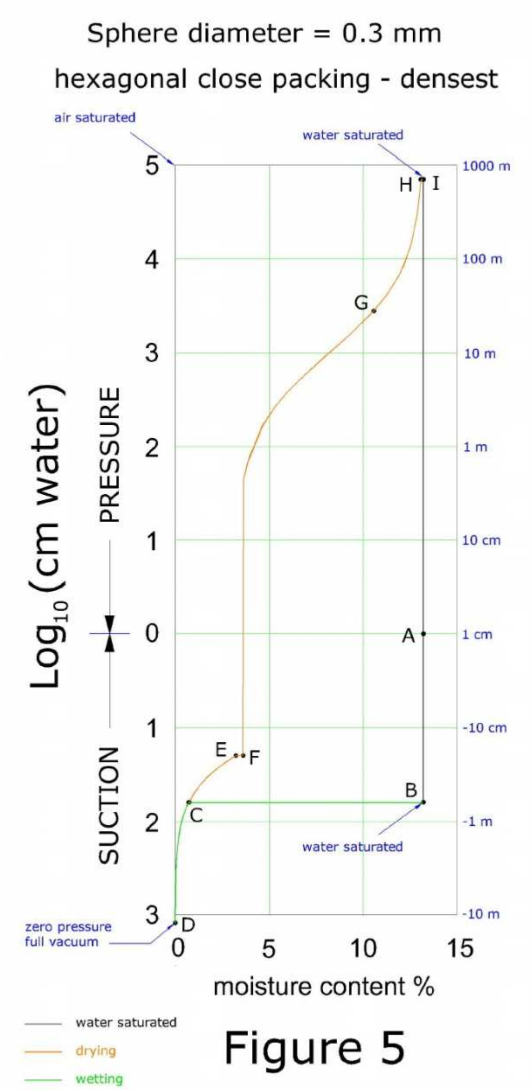

Figure 5

Point **G** is attained as ambient pressure increases, causing the volume of entrapped air to be further compressed, thereby allowing space for more water to enter the pores. Once **G** is reached the air volume can be contained within a simple sphere, one which can exist within the void space without itself being distorted by touching the solid phase.

Progressing to Point **H** is a continuation of the above volume compression under increasing ambient pressure. But once the conditions at **H** have been achieved the air bubble is sufficiently small that it can escape its entrapment within the voids by passing through the space available between the solids.

Point **I** is attained as a result of the air venting once **H** has been reached, thereby completing the wetting cycle and the soil becoming a water saturated 2-phase material again. This system cannot become air-entrained again without being subjected to the conditions attending Point **B**, and thereafter starting into another drying cycle.

Figure 5 represents exactly the same pattern of behaviour for the same sized assembly of sphere, but in this case, arranged in the densest packing. Both figures are drawn to the same scales, the relative narrowness of the dense packing reflects the smaller void spaces available for water &/or air.

7. Water Retention within the Solid Aggregation

The locus of any/all viable instances of geotechnical 3-phases systems is constrained to fall somewhere or other on the lines bounding the hysteresis confinement; there is no other physically compatible placement at normal groundwater temperatures. The drying cycle is controlled initially by exerting suction on the ambient (exterior) water, then by evaporation. The wetting cycle is advanced initially by condensation, then after collapse a **E**, by increasing ambient water pressure.2

The important geometric relationship governing the idealized aggregations dealt with herein is that of the maximum spherical opening which exists between the solid spheres, that space through which an air bubble can either enter or exit the void spaces. This is equal to $(\sqrt{2}-1)D$ for the loose packing and $(2/\sqrt{3}-1)D$ for dense arrays, where D is the diameter of the solid spheres. This opening size controls both the initial air entry at the start of the drying stage (**B**), and again at the end of the wetting stage (**H**) to permit the release of the air from the system.

At both extremes of drying and wetting it is worth noting the following: At the dry extreme (**D**) we encounter a situation which bears on the survival of humanity, because when suction exceeds 10m we pass the point where plants can any longer suck/draw water from the soil – what in agriculture is called the "permanent wilting point". Then, at the top of the hysteresis we see that in order to purge the system entirely of entrained air, very high ambient pressures are theoretically needed - about 100m in the case of loose arrays and more than 700m for the dense. This latter pressure is for all practical purposes simply not attainable.

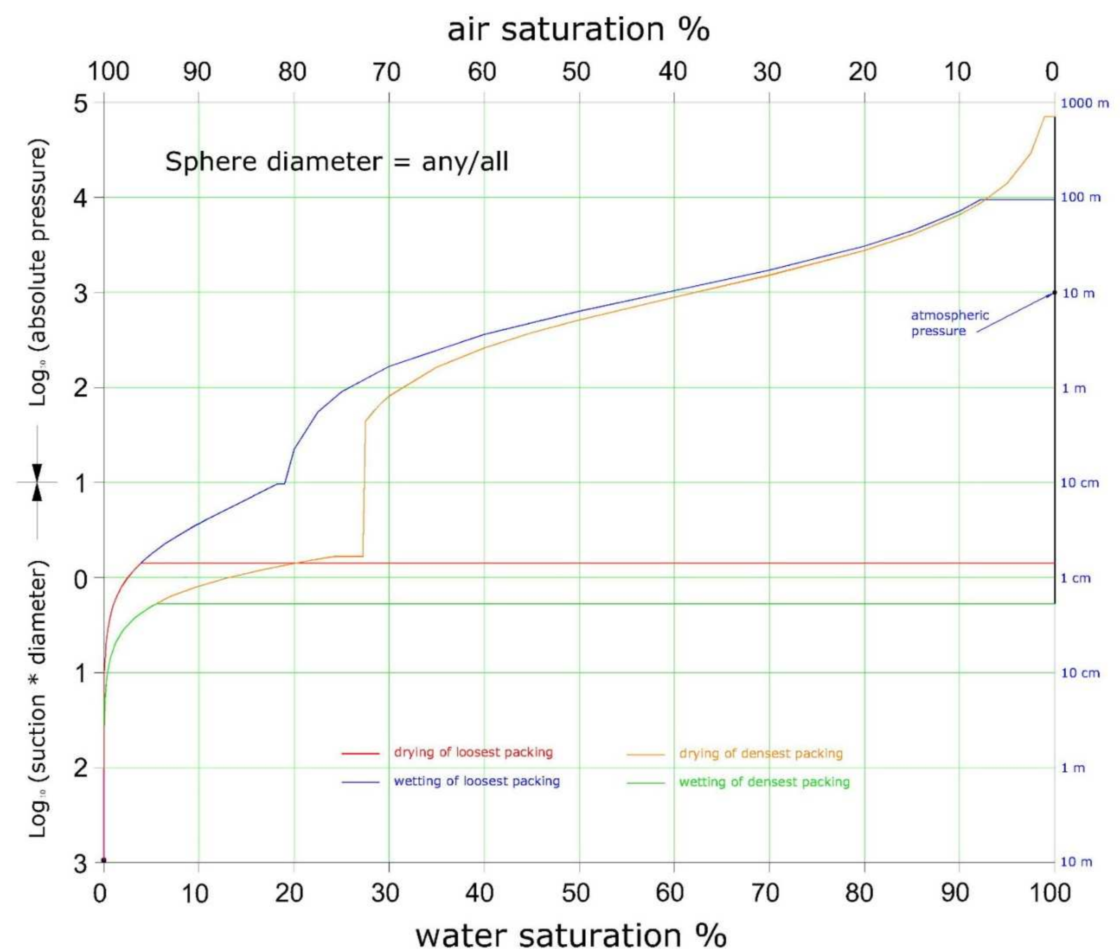

Figure 6

In order to summarize the hysteresis curves for both the loose and dense states already discussed, and furthermore, to provide this same information for any size particle, rather than confining it to only 0.3mm diameter, Figure 6 has been constructed. To accomplish this it was necessary to find a way of normalizing both axes. The abscissa was easy since changing from moisture content to degree of saturation did that. For the ordinate scale, the pF used up till now to measure suction was set at the logarithmic product of suction and diameter. This works because suction is a direct function of particle size. Similarly, the pressure scale is set at the simple logarithm of absolute pressure since void-confined air volume is not related to particle sizes.

The normalized axes of Figure 6 help to show that the area within the hysteresis curves is a function of work/energy since "pressure by volume", is equivalent to "force by distance", both carrying the dimensions of energy.

8. Permeability of Aggregation

Entrained air bubbles impede water flow through the pores of the soil and consequently reduce water permeability (hydraulic conductivity), and they do so to the same degree as would a solid particle of the same size. So the permeability of a 3-phase system depends upon the locus its state occupies on the wetting-drying curve.

Between Points **I** and **B** the system is fully saturated (2-phase) and permeability is at its maximum because all of the pore space is available for water flow. Beyond **C** in the drying cycle permeability is completely lost, and remains zero all the way past **E.** This is because there is no water continuity within the pores, only isolated annuli of menisci attached at particle contacts. It is not till the collapse of **E** to **F** that some water continuity is restored. Thereafter it gradually increases as the volume of void air is progressively squeezed smaller by increasing ambient water pressure. Finally, when the 3-phased system arrives at Point **H** the air bubble has been forced small enough to escape from the pore space, allowing replacement water to enter thereby attaining the 2-phase state again at **I**. It should be noted that in the 2-phased state the ambient water pressure has no effect on permeability and can be increased to any level. And provided it does not fall to Point **B** (which would return the system to a 3-phase state) permeability remains at its maximum value.

For my own use I have written a computer program to estimate hydraulic conductivity; the source coding is freely available at Reference 1. This program employs user-supplied input values for soil-structure void ratio, particle size distribution, hydraulic gradient, and water temperature in its computations. It uses the J.S. Kozeny inspired technique whereby an equivalent pipe diameter can be assigned to any soil aggregation by equating the Fluid Mechanics parameter, hydraulic radius, to the Soil Mechanics' ratio of pore volume to surface area of all the grains. Once in the pipe-analogy mode it is a simple matter to determine conductivity from a combination of the Darcy–Weisback formula and the Colebrook equations for surface roughness (e/D = 0.05 adopted herein). Flow types ranging from laminar to turbulent are then automatically assigned appropriate parameters based on their Reynolds Number.

The presence of entrained air bubbles can be accommodated by entering bubble sizes and frequencies into the particle size distribution as if these bubbles were solid particles. The equivalent particle diameter D_{AIR} can be calculated from their prevailing system locus/position on Figure 5 as follows: $D_{AIR} = D\ (e\ \ A_{SAT})^{1/3}$, where A_{SAT} is the degree of air saturation expressed as a decimal quantity.

9. Apparent Cohesion

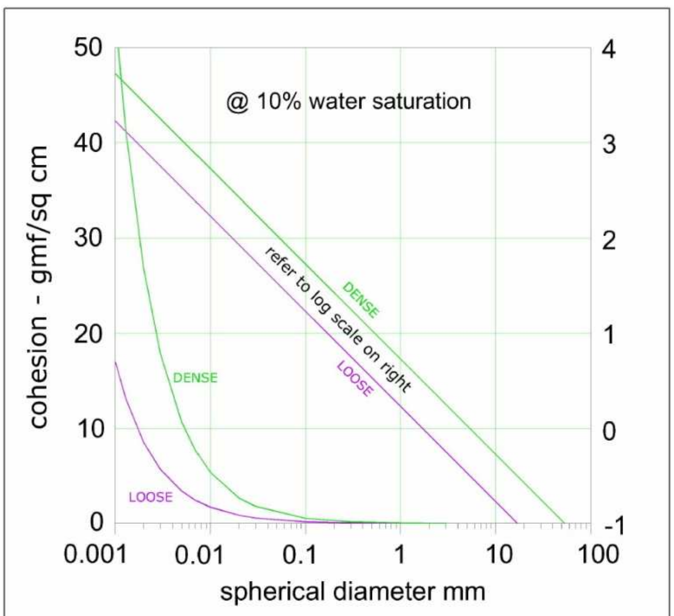

Figure 7

Pure water has high tensile strength. This is obvious from the height to which trees can feed water from root level to their leaves, with some trees in the Redwood National Park, California having been measured taller than 100m. Under ideal laboratory conditions water has been shown to have a maximum tensile strength of about 3,000m of head. But any impurity or water bubbles in the column will greatly diminish this value, as is attested to by the 5m limit to which suction pumps are useful/reliable.

It must be emphasised that the cohesion being considered here is not the permanent kind that comes from chemical or adsorbed water bonding between naturally occurring soils. Rather, the following refers to the "apparent" cohesion (C_A) which depends on the presence of menisci between particles within a partially saturated soil as depicted in Figure 2. This cohesion is ephemeral inasmuch as it disappears when/if the soil becomes too wet.

The equations relating to menisci listed in Section 3 allow C_A to be calculated. Both the surface tension of the water/air interface together with the relative pressure deficiency in the liquid phase contribute to a tension between touching particles. That tensile force, tending to pull the particles together, can likewise be appreciated as a cohesive force resisting them being pulled apart. And in this context we call such a resistance "cohesion".

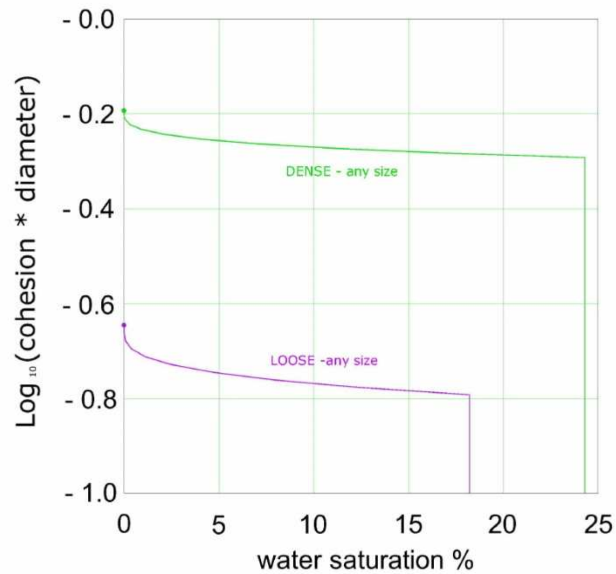

Figure 8

Before being able to speak of cohesion as a stress/strength parameter, for instance as it is shown in Figure 7, it is necessary to stick with it in terms of a force vector a while longer. This is because in the case of loose (cubic open) packing each particle touches 6 neighbours, and for the dense (hexagonal close) packing the number of interparticle contacts is 12. Since each contact has an attached meniscus we must resolve these several forces and find the resultant, and that, in the direction we are interested. Afterwards we are able to slip into thinking in terms of stress, by dividing the resultant by the area the particle occupies on the plane orthogonal (normal) to the force. The data points behind the curves drawn in Figures 7 and 8 were computed accordingly.

Figure 7 shown here relates the values of apparent cohesion plotted against particle size. Here cohesion is quoted in terms of stress, using the dimension gmf/cm^2 as elsewhere within this document. These data are for the particular case of 10% water saturation. As a generality three things are obvious: cohesion is greater for denser packing; cohesion increases rapidly with decreasing particle size; cohesion is of much more significant in silts (<0.06mm) than in sands. Figure 8 plots the same data against normalized axes in order to present values for any particle size as well as for the full range of water saturation.

It needs to be noted that interparticle menisci can exist only for the conditions that prevail between the Points **D** to **E**, thus C_A cannot be generated outside/beyond that stretch of the hysteresis curve. Somewhat counter intuitively, the highest cohesion does not reside where the menisci are most voluminous - it exists close to the point of almost complete dryness (near **D**). The qualifier "almost" is required because although greater C_A values occur when R1 & R2 are smallest, obviously when there is no water there can be no cohesion.

In their designs, geotechnical engineers quite rightly take the position that placing any reliance on C_A would constitute an unwarranted risk, and in consequence, assume the ground is saturated, that being the ground's weakest state from the perspective of shear strength. However, this "sensible" assumption unintentionally brushed aside the sometimes undesirable implications/consequences inherent in making unsaturated (3-phase) soils much less predictable and manageable in practice. This issue is addressed in the following section.

10. Consequence to Earthfill and Tailings dams

Although the following discussion refer to the post-construction behaviour of two large hydroelectric dams, much the same principles and consequences are applicable to tailings dams. In the latter case there is more reason to be fearful, because these are built for speculative mining operators who generally cannot predict their final dimension . . . they are always a work-in-progress. In comparison the dams owned and operated by public utilities are built in one continuous sequence and made to endure, and also get the supervision, monitoring and maintenance they warrant.

Earthfill embankment dams can be the source of high water pressures within the pore spaces of the soil-structure. And since the earthfill is normally placed and compacted in an unsaturated state such pressures can be an important instance of a 3-phase material. As soon as the reservoir begins to be impounded, and water invades the fill, the forces of cohesion between individual smaller particles inevitably change to some extent here and there throughout the dam.

Common practice in earthdam embankment construction is to ensure the earthfill is partially dry when placed ready for compaction/densification. The target placement moisture content for the non-plastic (discrete particle) types of fill modelled theoretically herein is typically "2% dry of optimum". Optimum here refers to that moisture content which has been determined at the site laboratory to yield the highest packing density for that specific material, and optimum itself falls about 5% short of water saturation. Therefore, these earthfills start out between Points **D** and **E** on the hysteresis curve, both because they are not saturated, and no ambient water pressure is applied.

Compacting the fill in a moist (3-phase) state has the advantage that interparticle cohesion facilitates the compactive effort by restraining the otherwise free movements of grains. On the other hand it renders the fill vulnerable to some degree of soil-structure collapse and readjustment if subsequently, after reservoir impoundment, conditions come to prevail where the system moves past Point **E**, towards Point **G**, whereupon all cohesion would be lost.

Two field cases are now discussed where characteristics of the 3-phase model behaviour are applied to earthdams in order to see if they can help understand post-construction behaviours, manifested as surface depressions, which caused major fears about the stability of these world-class structures. Tarbela dam on the Indus River in Pakistan's NWFP will be used to illustrate upstream problems; Bennett dam on the Peace River in western Canada for downstream problems.

10.1 Upstream problems

During my residence there, Tarbela Dam was the largest earthfill dam in the world – and surely it was also the most troubled. The earthworks were instrumented, and monitoring was conscientious. After several years of attempting to fill the reservoir it was noticed that the piezometric surface across the "impervious core" did not conform to the design intent. Rather than the pore water pressure gradually reducing from reservoir level at its upstream side of the dam to tail-water level on the downstream side, there was no perceptible loss across the wetted extent of the core. So, piezometers close to its downstream side were registering the full, undiminished, hydrostatic force of the reservoir.

Credit must be given to John Lowe III, geotechnical engineer and senior partner of TAMS NY who designed the dam. He correctly diagnosed this otherwise fearful condition as being a natural consequence of the core actually being effectively impervious to water. Therefore, the situation being that no reservoir water was flowing through the core, the wetted upstream part of the core, quite properly, showed no piezometric losses; while the downstream part was still almost dry. He attributed the impervious nature of the dry part of the core to air entrainment within its pore spaces making that material essentially impervious to water. This explanation allayed the fears in this specific regard.

The upstream face of a high earthdam is a good instance of 3-phase soil behaviour under large ambient pressure reversals. Each year the reservoir level changes, often by a matter 100 metres or more, from full pond to dead storage. Depending on soil gradation and elevation within the embankment this could well be enough to put pores through their full wetting-drying cycle year after year. Suffering such repetitions of loss and reestablishment of cohesive forces, it would not be surprising if there was a significant re-aggregation of the discrete particles from which the fill is composed. Under such circumstance, however well the earthfill had been compacted, it must be expected that somewhere or other in the millions of cubic metres of earthfill, pockets of loosened fill might align themselves to make for a preferred seepage channel.

When a sinkhole appeared on the upstream face of Tarbela at about 50m below water level the reservoir was dumped. To fully appreciate the gravity of this situation coloured photographs help: Reference 3 shows several. Despite the hundreds of sinkholes that had already appeared in the upstream impervious blanket, it was thought that the main embankment itself would not suffer one. This expectation was based on the hypothesis that sinkhole vulnerability was related to the magnitude of differential hydraulic pressures exerted across the blanket; the higher the differential the less brittle the soil behaviour. As the differential was greatest at the dam section it was thereby predicted that a sinkhole would be less likely in those more plastic materials thereabouts. Unfortunately, such proved not to be the case.

Subsequently, I have come to believe this sinkhole was precipitated by fill density reduction brought on by large swings round and about the hysteresis curve as the reservoir level rose and fell during its annual usage.

10.2 Downstream problems

Two depressions ("sinkholes") appeared on the crest of Bennett dam on the Peace River at Portage Mountain in British Columbia. Prior to this happening the core instrumentation indicated behaviour similar to what had been recorded some years before at Tarbela, a decades-long period when the wetted front moved ever so slowly downstream. This suggested to me an explanation, in fact two explanations, for the subsequent development of sinkholes/depressions downstream. Both conceptual mechanisms fit, better or worse, with what might be expected to happen quite naturally during the course of time within an earthfill, simply because of it being constructed of a multiphase material.

Both proposition had in common John Lowe's explanation that the advance of the wetted front was stalled by air entrapped in the voids ahead of the front. Now, as I perceive it, since the downstream unsaturated core material would have been a 3-phase material in a state somewhere between Points **D** and **E**, that material would have been virtually impervious, as John Lowe explained at Tarbela. So, in order to render that dry core water-permeable it's state had to move past Point **E**. And it is here that my two conceptual mechanisms differ: in the manner by which the front eventually manages to creep forward at all.

My first idea was that, at the immediate front, the water on the upstream water-saturated side, might take the void air on the downstream side into solution, thereby removing it as an obstacle to seepage flow. But in order to persist with this air removal activity it would be necessary for the water at the front to replenish its air-solution capacity. My thought at that time was this could only be done by dissipation: passing the air from the front back to the reservoir. This would maintain some level of receptive, "empty" water at the face to continue the work. Of course this would take a lot of time, but then, that was one of the characteristics that needed to be accounted for anyway. Apparently, the design team gave this notion enough credence to subsequently publish this concept in *Geotechnique*, or so my colleagues told me. However, now that I have given more thought to this phenomenon, I have come to think that old idea of mine was wrongheaded.

My second, and current idea, is that the frontal advance would more likely have been accomplished by differential vapour pressures at the interface, whereby evaporation from the water saturated pores on the u/s side would have condensed on the menisci downstream. Thereafter, once Point **E** had been reached in this way, the high (full reservoir) pressure front could exert its influence by compressing the void air volume still remaining on the downstream side, pushing towards Points **H** and **I**, and all the while making a greater expanse of the core more permeable to water. This, of course, being also a slow process would similarly explain the time-lapse phenomenon characteristic of the experience on both dams.

The eventual breakthrough of the wetted front to the downstream face of the core would then allow the onset of steady-state-seepage flow. Once that happened, the downstream side of the embankment would come to experience drainage water for the first time. And since the two depressions at Bennett dam developed coincident with the positions of two vertical settlement gauge pipes, it is altogether possible the whole incident could be accounted for by the backfilling around those vertical pipes getting wet, and the attendant new cohesive forces causing contraction of that fill material. It is usually the case that fill closely surrounding instrumentation does not get the same degree of compaction as the surrounding bulk of the shell fill itself, and in consequence, it is more vulnerable to volume changes.

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11. References

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