Compacted Earthfill is a 3-phase Material

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ABSTRACT

The purpose of this article is to pull together those facets/aspects of Soil Physics which are pertinent to civil engineering structures in order to make for their better design based on principles not taught to geotechnical engineers. These are primarily about how the 3-phased nature of the partially saturated materials used in earthfill embankments can eventually lead to the development of serious post-construction problems. Two high-profile hydroelectric dams, Tarbela and Bennett, are examined in this regard, and it is shown how the unsatisfactory behavior of their upstream and downstream slopes are directly traceable to the 3-phased nature of their earthfills. Mathematical formulations are suggested in order to provide rough valuations for permeability (hydraulic conductivity), and for apparent cohesion, of such unsaturated materials.

Article 3 . . .

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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 kinematics of inter-particulate contacts, could be useful. It is in this spirit that the following ideas are put forward.

The approach adopted here is to simplify this intractable problem by modeling 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. This position is freely acknowledged, but then, whatever else can be done to move above 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 emphasize 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 the Royal Society, and also in his 1969 *Geotechnique* article "Osborne Reynolds and Dilatancy". This approach seems to have gone out of fashion, probably because later generations, supposed such eminent engineers/thinkers must have exhausted this particular means towards enlightenment.

Anyway, the plain fact of the matter is that in order to make any advance in practical 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 idealization of behavior.

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

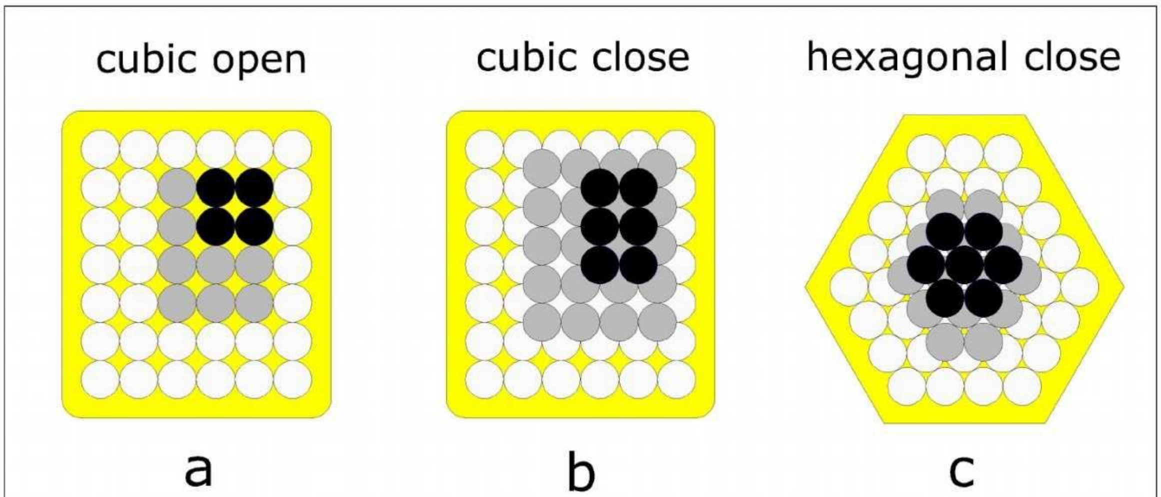

Figure 1

MENISCUS FORCES & VOLUMES

Having conceded the theoretical need to adopt uniformly sized spherical shapes in regular arrays as a first step, it then becomes possible to attempt making a determination of the idealistic principles which govern earthfill behavior.

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.

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 vapor phases. The following set of equations allow calculation ofthe 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 spherical particle

T = 0.0741 N/m at 10°C.

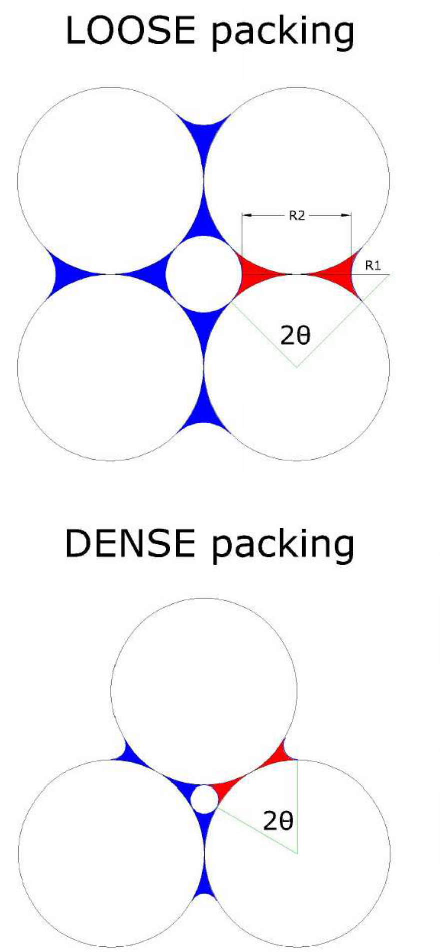

Figure 2

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 (Ref. 1), together with Boyle's Law allow the full drying-wetting hysteresis curves, shown in what followings, to be drawn.

DRAINAGE OF PORE WATER FROM VOIDS

One of the most important lessons geotechnical engineers can 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 here 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 the phenomenon is referred to as the "ink-bottle effect". This allusion is 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 remain full until some air is allowed to enter 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. Next, 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 can't get out to let the water in. Kindergarten-esque though this experiment appears it nevertheless illustrates the physical principle which prevails in the void spaces within an unsaturated particulate mass. It's easier to empty the voids than to refill them.

The results shown on Figure 3 are from laboratory work detailed in Ref. 2. The Soil Physics unit/measure of suction (sub-atmospheric pressure), use 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, vapor 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.

The test results from the coarse specimen show a gradual water loss from ambient pressure down to 1pF (= suction 1), 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 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 4pF. The equivalent circular pore diameters corresponding to these suctions are 0.01 mm and 0.003 mm. The moisture retention characteristics of the mixture shows 88% of the water was lost between 1.5pF and 4pF. 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.

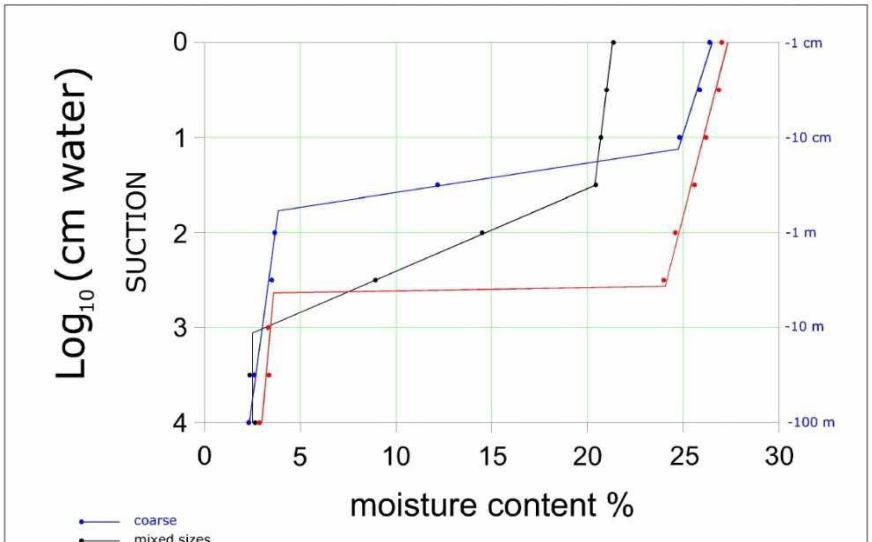

Figure 3

FILLING DRY VOIDS WITH WATER

Back in the late 60s 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 process of rewetting the dried out pores. Presumably this was because for those working in Soil Physics research there was little of practical interest in that aspect of soil behaviour.

On the other hand it seemed that it might be of value to civil engineering to complete the cycle – to close the loop of drying and rewetting of the voids within the soil-structure. Basically, to see if this phenomenon had any bearing on how engineers go about their business using Soil Mechanics.

With that aim in mind the closed curves shown in Figures 4 and 5 were constructed to represent the loci of a continuous relationship between moisture content and ambient water pressure. With that accomplished it is now possible to 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. The abscissa with its measures of either water content or percent saturation are quite familiar, but the ordinate scale needs some introduction.

A choice was made to use a compound-logarithmic ordinate scale for the following reason: Because of the great spread in the range of pressure data of interest, a logarithmic scale is necessary to cover the full scope of relevant 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.

PROGRESSING AROUND THE HYSTERESIS LOOP

Figure 4 is a graphical representation of the full drying - wetting 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 with regard to earthquake liquefaction. In the generality of multi-phase physics the void space between the solid particles can be occupied by either liquid or vapor, or some combination of both. In geotechnical engineering the liquid is water and the vapor is air.

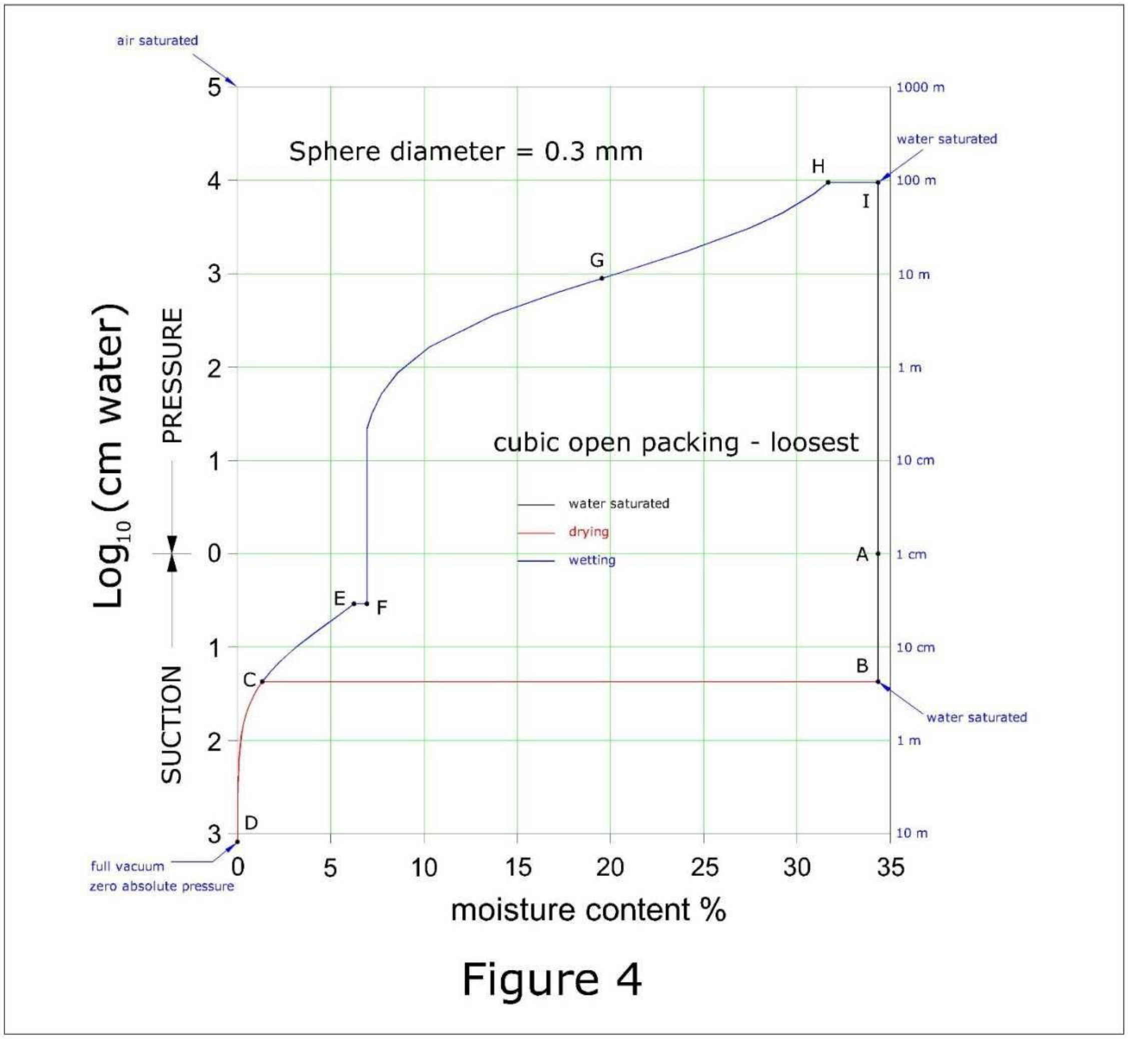

Figure 4

Point **A** is the beginning and end of the loop where the soil is fully saturated at ambient pressure and the voids are filled with water.

Point **B** is reached 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 thereby passes from a 2-phase to a 3-phase material.

At Point **C** the loss of water comes to a halt, and the moisture content cannot be reduced further by depressing the ambient pressure. 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: permeability is completely lost. The only water within the pores is in the form of isolated annuli of menisci attached at particle contacts.

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 vapor into the menisci, thereby increasing the moisture content while lessening the suction level (increasing the pore 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. The only physically plausible outcome under the conditions that pertained at this stage, in order to permit further wetting, is a catastrophic inversion of the air/water interfaces.

At Point **E** the combined areas of the menisci surfaces are more than that required to enclose the existing void air in a single containment. Since surface tension carries the dimensional units of energy/area (for example J/m^2), the minimization of the area of envelopment is the prevailing imperative. The physical solution to this impediment is for the menisci surfaces to be instantaneously reconfigured into another shape, one with lesser area. This shape is basically a modified (non-spherical) bubble where the roundness of bubble is disfigured by some intrusion of the surrounding solids. This event involves the complete reversal of surfaces role, from one of constraining the water against the particles, to one of containing the void air within a single envelope. During the implosion involved, there is an energy release, and it is the work that can be done by the newly available energy that explains the next position, **F,** along the wetting cycle.

Point **F** is calculated on the understanding that the excess energy released by the collapse of **E** 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 exist at a slightly higher pore water content appropriately removed from **E**.

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 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.

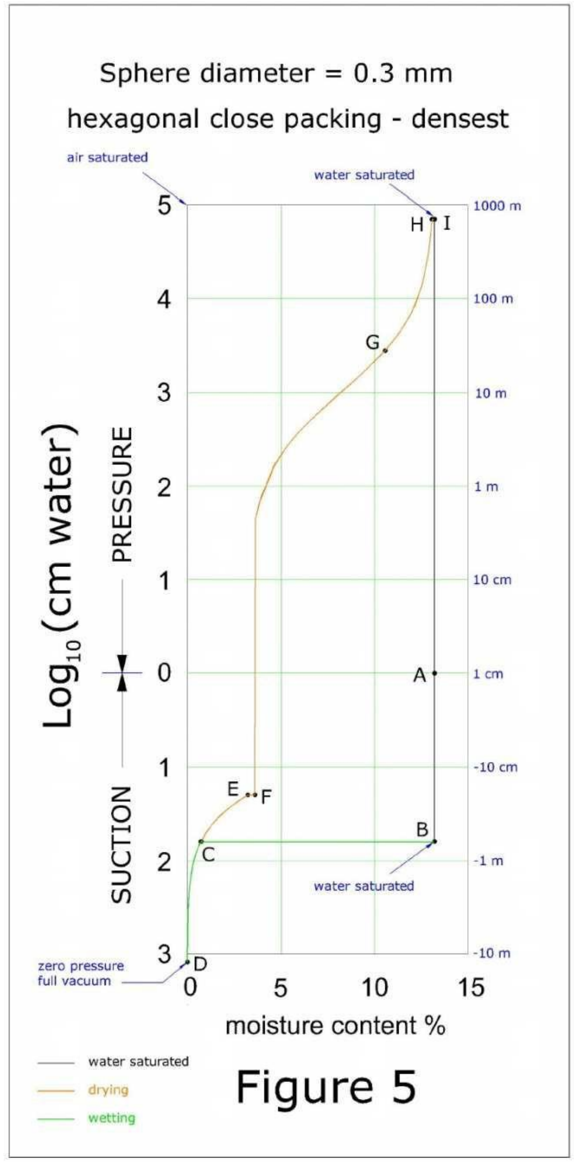

Figure 5

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.

WATER RETENTION WITHIN AGGREGATION OF SOLIDS

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 from **E** to **F**, by increasing ambient water pressure.

The important geometric measure governing the idealized aggregations dealt with herein is that of the maximum spherical opening which exists between the solid spheres, that through which an air bubble can either enteror 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.

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”, being equivalent to “force by distance” carries the dimensions of energy.

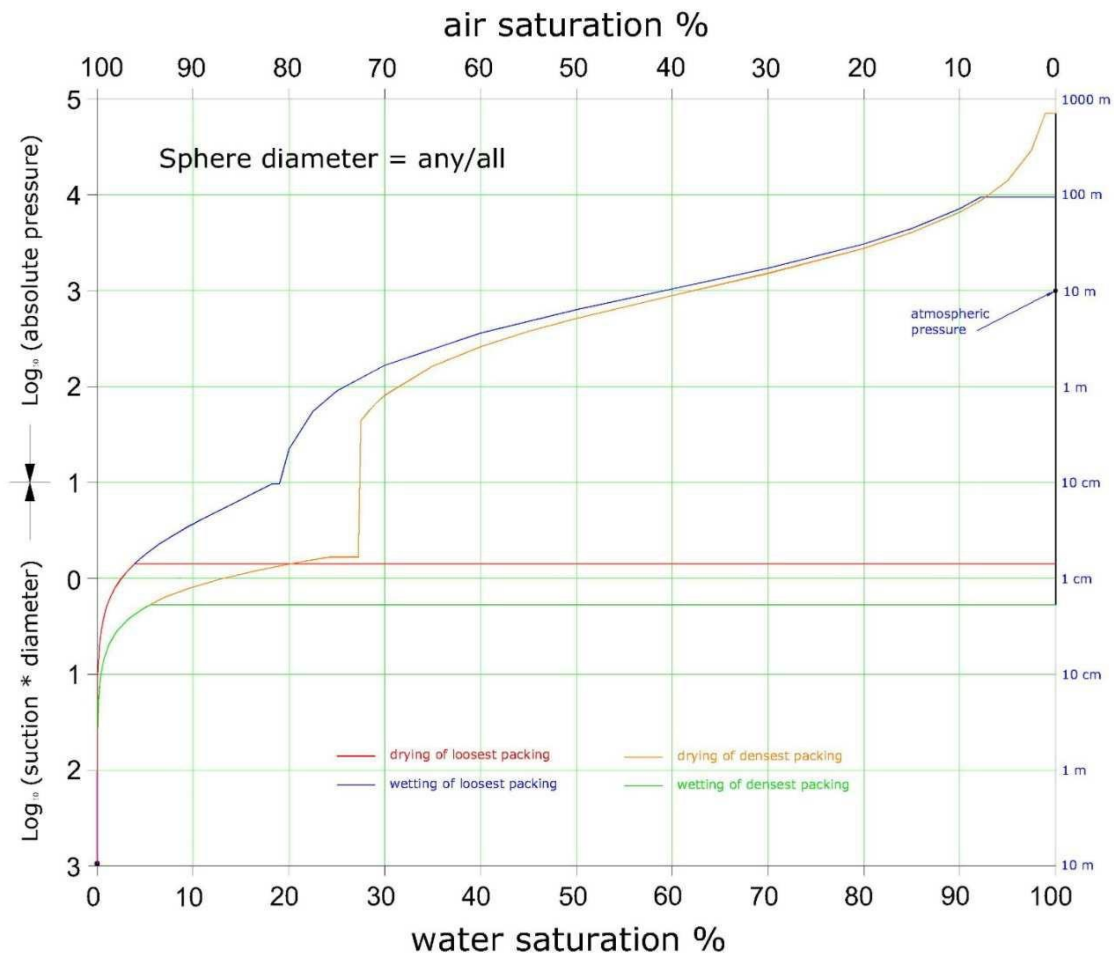

Figure 6

However, it must be acknowledged that this procedure involves a compromise with mathematical propriety: Since the antilog of zero is 1, that means that the zero-ordinate has the value 1cm H_2O pressure on the top side of the line, whereas the same line, on the suction side representing 1 cm H_2O of negative pressure. Therefore, a line which should have no thickness, here turns out to be 2 cm wide. Nevertheless, it was decided to gloss over this mathematical nicety in favor of clarity of depiction.

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 drying-wetting 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 a somewhat higher ambient pressure after the collapse of **E** to **F,** and on its progression towards G, that water continuity is finally restored. Thereafter it gradually increases as the volume of void air is progressively squeezed smaller by any increase in the ambient 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 attain 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.

A computer program has been written to estimate hydraulic conductivity; the source coding is freely available at Ref. 3. 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). Flow types ranging from laminar to turbulent are then 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. These 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. It needs to be kept in mind that air bubbles differ from solid particles inasmuch as bubble size is subject to ambient pressure.

APPARENT COHESION

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 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.

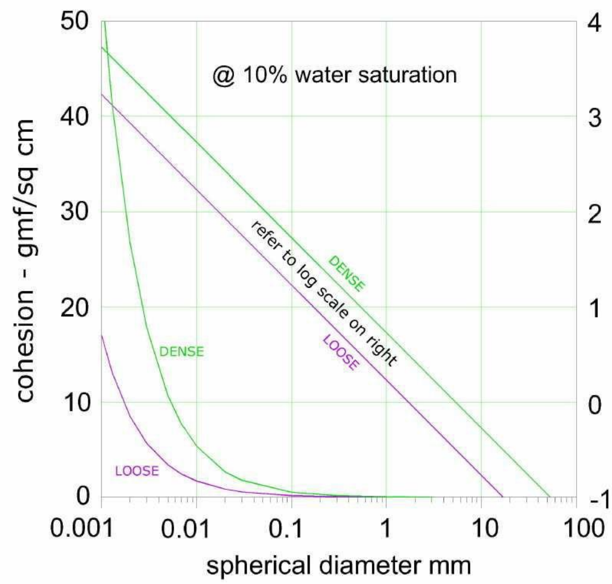

Figure 7

It must be emphasized 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) that 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 saturated with water.

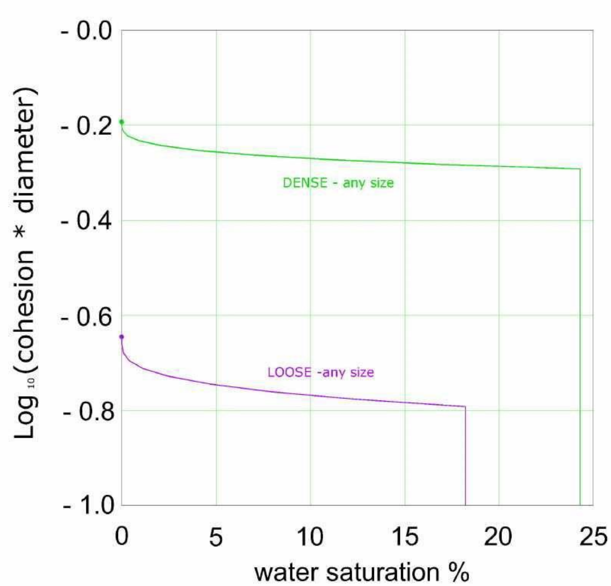

Figure 8

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 in.

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 (g/cm^2) as elsewhere within this document. These data are for the particular case of 10% water saturation. As a generality, three things areobvious: 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 data for any particle size and 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 near the point of almost complete dryness (near **D**). The qualifier "almost" is required because although maximum C_A occurs when R1 & R2 are minimum, 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 brushes aside the sometime undesirable implications/consequences of making unsaturated (3-phase) soils much less predictable and manageable in practice. This issue is addressed in the following section.

CONSEQUENCES to EARTHFILL and TAILINGS DAMS

Although the following discussion refers 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 they 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 These structures also get the supervision, monitoring and maintenance they warrant.

Two field cases are now discussed where characteristics of the 3-phase model behavior are applied to earthdams in order to see if they can help understand post-construction behaviors, 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, and Bennett dam on the Peace River in western Canada will be used to illustrate upstream and downstream problems, respectively.

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But before getting into the particulars of site specifics it would perhaps be in order to mention some generalities considered normal in engineering construction.

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 modeled 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 **C** and **E** on the hysteresis curve, both because they are not saturated, and little 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. This will be known to anyone who has tried compacting dry sand. 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 Points **E** and **F**, towards Point **G**, whereupon all cohesion would be lost.

Upstream Slope

During the late 70s Tarbela Dam was the largest manmade structure 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 a sinkhole appeared on the upstream face of Tarbela at about 50m below water level. The reservoir, which was within 5m of being filled for the first time, was immediately dumped. To fully appreciate the gravity of this situation photographs help: Ref. 4 shows several images.

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 then-current hypothesis that sinkhole vulnerability was related to the magnitude of differential vertical hydraulic pressures exerted across the impervious 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 thereabouts. Unfortunately, such proved not to be the case. In hindsight it seems more likely this sinkhole was precipitated by fill density reduction brought on by large swings round and about the hysteresis loop as the reservoir level rose and fell during its annual cycle of water storage and water usage.

The upstream face of a high earthdam is a good instance of 3-phase soil behavior under large ambient pressure reversals. Each year the reservoir level changes, often by a matter 100m or more, from full pond to dead storage. Depending on soil gradation, and at what elevation it finds itself within the embankment, this could well be enough to put pores through their full drying-wetting cycle year after year. Being exposed to such repetitions of drainage and inundation it would be surprising if there were not some re-aggregation of the discrete particles from which the fill is composed, no matter however well the earthfill had been compacted.

As soon as the reservoir begins to be impounded, and water invades the coarser fill, the forces of cohesion between individual smaller particle can be extinguished. Finer fractions of the fill, having lost their bonding, might then fall between the pore spaces of the larger particles beneath. Under such circumstance it may be anticipated that, somewhere or other, in the millions of cubic meters of earthfill, pockets of loosened fill might find themselves aligned in such a way as to make for a preferred seepage channel. Thereafter, here and there, throughout the upstream slope, the possibility has been opened for the eventual development of a sinkhole.

Stalling of Core Saturation

A second cause for concern at Tarbela was that the piezometric head across the upstream sloping core did not conform to the normal design assumptions. 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 energy loss across the wetted extent of the core. So that piezometers close to its downstream side were registering the full, undiminished, 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 downstream, dry part of the core, behaving as if it were virtually impervious to water flow. He attributed this lack of permeability to air-entrainment within its pore spaces. Under such circumstances, the wetted upstream part of the core, quite properly, showed no piezometric losses because no reservoir water was flowing through it. And this logical explanation allayed the fears in this specific regard.

Looking at the piezometric evidence it must be acknowledged to be a “snapshot” of a phenomenon which had a history of development towards the situation, there and then, apparent. So there remained the question as to why this process took such a long time to progress downstream, especially since there was a substantial hydraulic pressure gradient across the interface between of the wet and dry earthfills.

Of the several mechanisms which have been postulated to explain the very slow deployment of the phreatic surface within the downstream shell, the following seems to be the most credible/defensible. It is based on a recollection of the actual climatic conditions which prevailed at this particular site. The argument goes as follows:

There was a large thermal difference between the near-freezing reservoir water and the embankment fill which was quite warm, perhaps about 30°C. Therefore, on the upstream side of the interface the voids were water-saturated and at a high ambient pressure, but cold. On the downstream side the voids were much as those at Point **C**, that is, without moisture continuity, but hot. This is quite the reverse of a favorable, or neutral thermal gradient which would have facilitated transfer of water vapor downstream, and which would have accommodated saturation of the entire core in a more timely fashion. But the laws of thermodynamics do not permit this unfavorable thermal gradient to be reversed, nevertheless given enough time, it would allow this gradient to be negated.

The fact is that the soils immediately adjacent to the interface are in intimate physical contact across the divide. Therefore it is quite reasonable to accept that the downstream warm dry particles would locally transfer heat to the saturated soil upstream to the extent that, again locally, there would be no differential temperature and the adverse thermal gradient would have been equilibrated. Then, under this rectified situation, upstream diffusion and downstream condensation could take place, thereby allowing the process of downstream water-saturation to move an incremental distance ahead.

In time, steady after attaining condition **F**, the inflow of liquid water and the reservoir pressure will quickly push the remaining air bubble out of the void and it will become water-saturated. Eventually thereafter, once the rest of the core has expelled the air bubbles, the core becomes a 2-phase system where steady-state-seepage prevails, and the appropriate phreatic surface is established.

<u>Problems on Dam Crest</u>

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.

The foregoing explanation as to how the advance of the wetted front was stalled by air entrapped in the downstream voids seems appropriate for Bennett dam too. With the downstream unsaturated core material being a 3-phase material in a state somewhere betweenPoints **D** and **E**, that material would have behaved similarly to that at Tarbela. But given the northern Canadian climate in the case of Bennett the high adverse thermal gradient is unlikely to have been as great an impediment to progress of the evaporation-condensation method of saturation. In any case the equilibration mechanism cited above would have been available here as well.

When it came, the eventual breakthrough of the wetted front at the downstream face of the core would then have allowed 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 sinkholes/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 to be expected that fill closely surrounding instrumentation does not get the same degree of compaction as the shell fill itself, and in consequence, it is more vulnerable to volume changes.

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