the Water in the Soil – Part 5

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In the earlier articles of this series the talk was mainly about idealized particles, and it wasn't until Part 4 that real soils (sands) entered the argument. As a practicing Geotechnical Engineer, idealization is of passing interest: If it can't be used in the field [practice] it's really irrelevant. And, so saying, it's now time to move from contemplating single solid spheres and advance into the confusing realm of natural soils.

The key to making that move is what I've called the Crowding Factor, with the label "K". The reason for giving it this name is that its function is to account for all the hydrodynamic differences between the magnitude of drag forces exerted on a single solid particle moving against free/open water, and the same particle interacting with the much restricted pore water within the confines of a soil-structure void space.

What the Crowding Factor needs to do is to make it possible to take what we can learn from Fluid Mechanics and be able to use it to our benefit in Soil Mechanics.

Possible Ways of Evaluating "Crowding"

My initially thoughts on how to go about assigning values to this parameter "K" ranged from theoretical to empirical.

To start with, it seems pretty clear that what most changes in the immediate hydraulic environment of a particle, between its state as a single mass moving through boundless water, and its radically more confined state within a soil-structure, is the velocity of the water interacting with it. In the soil the water is speeded up while the particle's own velocity is not.

This suggests that for any approach to find justifiable values for the Crowding Factor the obvious target for manipulation is velocity. Here, it may be recalled, that both the Bearing [F_B] and Pressure [F_P] components of drag are functions of velocity, in the latter case, to the second power. Apart from the fairly fixed physical attributes of water, the only other significant variable in these components is particle size.

The first thing that came to mind was how we normally convert open water flow, the approach velocity [v_A], to the equivalent constricted pore space flow, the void velocity [v_V]. And that is simply to take it that for any given rate of flow the velocities are inversely proportional to the cross-sectional areas available to them. So where the void ratio of the soil mass is "e", we get the average void velocity by multiplying the approach velocity by (1+e)/e. For instance, if we were to apply this rule to the loosest (e=0.91) array of uniform spheres we would get a void velocity 2.1 times faster than the approach velocity; and, for the densest (e=0.35) packing that ratio would equal 3.9.

This simple calculation would suggest that in the loosest packing the crowding effect would increase the value of the Pressure component [F_P] by a factor of 4.4. This component, you may recall, is the one I associate with pore pressure generation, and which is proportional to the square of the velocity. The equivalent multiplier for the densest packing would be 14.9.

If it were not for the fact that the diameter "D" is also part of the F_B term I might have been tempted to just leave it there, that is, go on to assume void space was the only consideration. So, where to look next ?

The ConeTec cylinder was available to me, and as it had the capability of recording the water pressures in front of an object as it fell through a water column, the opportunity was there to measure the comparative effects of dropping an array of spheres rather than a single ball. The thought was to drop arrays of ball bearings while recording the pressure front as the composite mass approached the transducers implanted in the base of the cylinder.

By running a series of tests, where the results of various array geometries and spherical sizes could be compared with the theoretical drag forces for that particular particle size, the Crowding Factor would be known for that case. It is obvious that a great deal of testing might be required to produce useful answers, and these data would for practical reasons cover only manageable sizes such as fine to coarse gravels. Silts and sands would be out of the question because of the minute size of the individual elements of the array. Another practical difficulty in this research venture would have been the unavoidable effect the housing (containing the array) would have on the data, and then, how on earth could a means be found to abstract that influence.

While I was grappling with these experimental difficulties it dawned on me that what I was really trying to measure was nothing other than what is elsewhere known as the Seepage Force. And this Seepage Force [S_F] could much more easily be determined in a standard laboratory permeameter. In the permeameter the problem of housing effects, and the all but insurmountable difficulties in testing smaller sizes, would not exist. I should now explain what is meant by S_F.

Seepage Force

Many years ago I came across the term Seepage Force in Donald W. Taylor's 1948 MIT textbook "*Fundamentals of Soil Mechanics* ". He showed that S_F per unit volume of saturated soil was the product of hydraulic gradient " i " and unit weight of water " γ_w", that is,

$$S_F \text{ / unit volume} = i\,\gamma_w$$

You can derive this formulation directly from consideration of the water forces and specimen geometry of a permeameter as follows:

Let the cross-sectional area of the soil specimen be "A" and it's length in the direction of water flow be "lgt". If "H_U" is the upstream (driving) head and "H_D" is the downstream (resisting) head, then the net water force (by definition, S_F) causing flow is ΔF,
where, $\Delta F = A\,(H_U - H_D)\,\gamma_w$.
Since the hydraulic gradient across the specimen is $i = (H_U - H_D) \div \text{lgt}$, and the soil volume is A times lgt, we find Taylor's equation as shown above.

In practice, I have found the S_F way of sizing-up the effect of water passing through soils quite useful. For those who may not be altogether familiar with the Seepage Force concept I'm going to take a slight detour which I think, apart from demonstrating that S_F is a real and significant phenomenon, should be of interest in its own right. This involves some testing my company conducted at the NRC hydraulic laboratories in Ottawa some time ago.

Model Testing at NRC Ottawa

During the 1980s, hydrocarbon exploration in the Canadian offshore Arctic used artificial islands built from locally dredged sand as drilling platforms. This involved pumping pipe-line dredge discharge into the shallow waters of the McKenzie Delta. This method of construction commonly resulted in side slopes as flat as 3° to 5° which ruled out their use in deeper waters because the enormous volumes of sand required to do this could not be placed within the time frame offered in the ice-free windows.

If steeper side slopes could be built, then the oil fields in deeper water would then be accessible. It seemed obvious to me that these flat slopes were the result of outward seepage flowing from the face of the accumulating sandfills. As I saw it, such destabilizing flows could be brought about by high pore pressures existing within the body of the growing islands as a result of the energy introduced into the soil-structure by the impinging slurry jet, as well as ongoing contractive distortions within the loose sand pile itself. So, if outward seepage was causing flat slopes, would inward seepage result in steeper slopes? Pumping water out of the sandfill while the dredge placement was progressing was maybe worth a try, - at least in the lab.

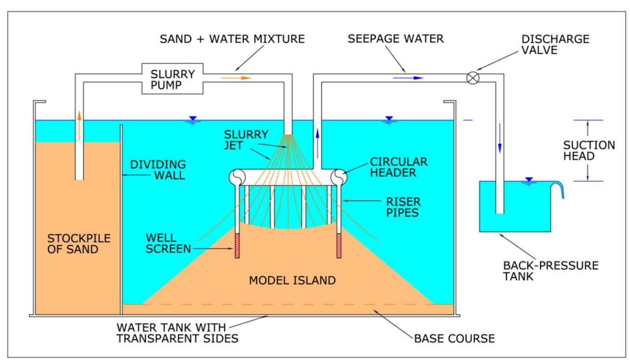

Figure 13: Schematic of NRC Ottawa model

Figure 13 is a schematic of the model we used in a series of tests done to see if the idea had any chance of working. Essentially, what is being checked here is whether Seepage Forces are real and potent, and whether they can be advantageously invoked by circulating water (in the right direction) through the underwater sand pile. The test setup employs a siphon to draw water from the inside of a sand pile at the same time as a sand slurry is building it up.

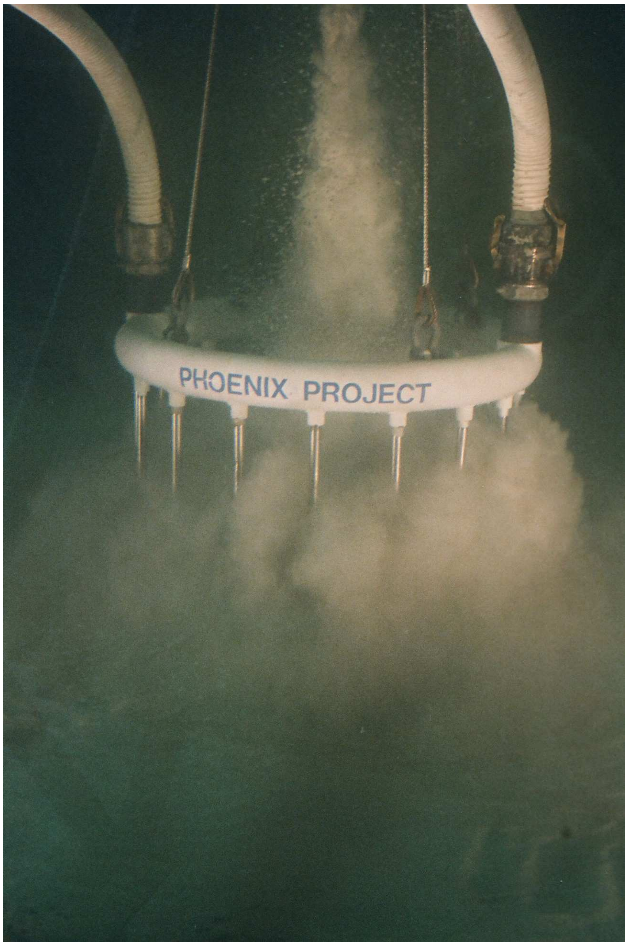

Figure 14a: Underwater sand slurry jet

Figure 14a is a photograph taken through the transparent front of the water tank showing the sand-water slurry jetting down through the water onto the space between the ring of well screens. Here it can be seen that the slurry has some features in common with liquefaction: individual sand grains having little, if any, solid contact with one another; surrounded by water; and, all moving energetically.

Figure 14b was taken during an interruption in sand placement. It can be seen that, at this stage, the sand is accumulating in a ring around the alignment of the well screens. This establishes that where the S_F is intense/concentrated enough to be potent, sand particles can be captured from the jet.

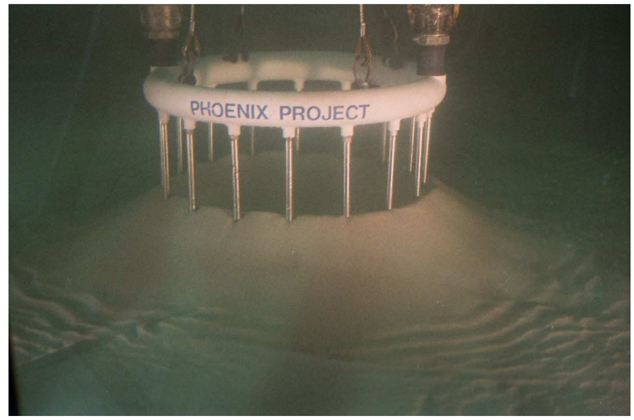

Figure 14b: Jetted sand accumulating around well screens

The Seepage Force is now acting in reverse (to its natural tendency). It is working to our advantage.

Figure 14c shows the sand pile which resulted after the wells had been progressively elevated (by gradually hoisting the circular header) as the slurry jetting continued. Average slopes of up to 38° were achieved, with slopes locally as steep as 45° nearer the well screens. These slopes, built dynamically in the abrasive environment of the impinging jet, significantly exceed the 29° submerged angle of repose achieved by gentle placement, but without the aid of an inward S_F.

Figure 14c: Steep underwater side slopes made by Seepage Force

Two important geotechnical forces are to be seen at work in these photographs and model results: the forces of Drag and Seepage.

1. Discrete sand particles jetting down the slope of the model are literally dragged into the face of the slope, and then secured in place, by the water velocity created across them by inwardly flowing water.

2. Otherwise overly-steep side slopes, of non-cohesive material, are made stable in a severe hydrodynamic context by the potency of the S_F as it pushes discrete particles into the face, thereby greatly increasing the effective normal stresses on them.

At a fundamental physical level these forces are closely coupled in their origin and influence, and perhaps should not be spoken of as separate behaviours. They are both a result of relative movement between the phases, in this case with the water doing the most of the moving.

Now that we have recruited the concept of the Seepage Force we can move on to building a bridge between the Drag Forces that can be calculated for a single particle, and those forces acting on the same particle size when it is just one among a multitude of particles of various sizes within a cramped and crowded soil-structure.

Defining the Crowding Factor K

The approach to both defining, and calculating, the Crowding Factor is as follows:

It is taken that the Seepage Force exerted on a given volume of saturated soil due to water flowing through it is a direct consequence and result of the summation of the Drag Forces exerted on its individual grains. Furthermore, the individual particle Drag Forces are taken as being equal to those proposed by Fluid Mechanics for spherical particles of equivalent size when exposed to the flow velocity existing within the voids of the soil-structure.

The value of the ratio between the water velocity in the void space [v_V], as compared to that of the approach flow [v_A], is K.

The definition of the Crowding Factor may therefore by stated as follows:

$$K = v_V \div v_A$$

such that if v_V is applied to the calculation of F_D, then the Drag Force per particle will be numerically equal to the S_F when v_A is used in the calculation of the Seepage Force for the soil (solid + water volume) associated with the same particle.

So the problem comes down to finding the factor by which the velocity term in the F_D equation must be multiplied to make the F_D force associated with a single particle equal to the S_F force for a single particle.

Theoretical / Idealized Approach

In order to give mathematical expression to the relationship between Seepage Force and Drag Force we must limit ourselves to dealing with spherical particles of uniform size.

By looking at a single particle and the volume occupied by that single particle we can write:

$$S_F = i\, \gamma_w\, (1+e)\, D^3\, \pi / 6$$

$$F_D = C_D\, \rho\, (v_V^2 /2)\, D^2\, \pi /4$$

In this particular instance I have chosen to temporarily revert to using C_D rather than using the component F_B and F_P, and this is simply for convenience: More mutual terms cancel out.

Now, setting $S_F = F_D$ and recalling that $v_A = i\, k$, we get

$$v_V^2 = 4\, g\, D\, (l+e)\, v_A = 3\, k\, C_D$$

which gives,

$$v_V + v_A = K =$$

$$= 2 \sqrt{(gD(1+e)=3\, k\, C_D\, v_A)}$$

This equation for K, it should be noted, requires an iterative process to recognize the fact that C_D, and in nonlaminar flow situations, k, are both functions of relative velocity. Such numerical awkwardness is avoided in the alternative approach outlined below.

The implication of the above mathematical derivation is that a value can be given to the Crowding Factor once the permeability of the soil has been established. Although I offer a theoretical solution for evaluating saturated soil permeability in the next article, it must be said that such solutions are at best approximations, and lab testing of good specimens is really the only way to go if there is any hope for accuracy in subsequent computations.

The above theoretical approach is useful inasmuch as it provides mathematical continuity to the overall hypothesis, however, the following approach is likely to be more useful in practice.

Empirical / Practical Approach

Earlier in this article I used the permeameter to help explain the Seepage Force. Now it would make sense to look again at this standard piece of laboratory equipment for an empirical solution to our current problem. What we can get from this tool is not only the permeability [*k*] needed to solve the above equation, but furthermore, we get a direct measurement of the actual Seepage Force exerted on the volume of soil comprising the specimen. And in fact, this is all we need to know in order to determine the value of K for whatever real soil, and degree of compaction, used to make the specimen.

How this is accomplished for a soil containing a range of particle sizes requires some explanation. Full details of this procedure, and a computer program to facilitate the calculations will be given in the next article. Suffice to say at this time, that what is involved is finding, by iteration, the unique value of v_v which will achieve the criterion that the summation of the individual Drag Forces on the particles within the mass should equal the Seepage Force for that volume of soil.

Although the permeameter is a standard piece of equipment in geotechnical labs, my preference for this particular investigation is for using the triaxial apparatus instead. There are four reasons for this choice:

1. Triaxial technicians are familiar with constructing specimens to explicit specifications and they know how to saturate and de-air soils. Air entrained in an otherwise saturated soil would artificially decrease the measured permeability and increase the Seepage Force.

2. The flexible membrane in which the specimen is enclosed provides a good boundary for the outer soil particles once the cell pressure exceeds the pore water pressure. A rigid (metal or glass cylinder) encasement of soil results in significantly higher void spaces around the specimen perimeter and this leads to artificially high values of permeability and lower Seepage Forces. This is particularly important in coarse uniformly graded materials such as can be tested in the large diameter setups available to us nowadays.

3. After the permeability and Seepage Force have been determined in the drained-mode the specimen can then be strained to see whether the soil tends to contract or dilate. This tells us whether deformation of the soil modeled in the test specimen will lead to increases or decreases in pore water pressure.

4. It is a simple matter at this stage to perform a routine drained or undrained compression test at the deformation rate of interest.

So where are we now ?

Fluid Mechanics and Hunter Rouse have given us access to hydrodynamic aspects of water flow at various velocities around spherical particles of various diameters, and that allows us to separate such energy flow losses into those which create water pressure and those (viscous) which do not. The visit to Fluid Mechanics also gave us a way of looking at liquefaction and the idea that the structural collapse / fall came before the pore pressure rise. Following this valuable excursion into Fluid Mechanics, it is appropriate to return to Soil Mechanics once it comes down to non-discrete particles in crowded assemblies, and to those aspects of soil-structure and agglomerations which geotechnical engineering is all about. I believe the combination of these sister disciplines gives us the best of both worlds.

In the Last Article

The method I propose for calculating pore water pressure changes generated in any gradation of saturated granular soil under deformation will be detailed.

I will make some general statements about what I believe to be the most important facts about the water in the soil.