Interpretation of Piezometric Traces by h-Method

William E Hodge

ABSTRACT

This is the first of three articles written with the intent of putting forward a coherent method to explain field experience by combining some of the hydrodynamic aspects of Fluid Mechanics with Soil Mechanics, into what is called herein the hydrodynamic method (h-Method). The first two articles deal with a 2-phase water-saturated soil: first, where the soil-structure forces its way against the pore water; then, where energized water works its way through the soil-structure thereby tending to destabilize the mass. The final article introduces elements of Soil Physics to explain how, in a 3-phase soil (solids, moisture and vapor) the freedom of water is powerfully constrained when it comes to moving into a body of fine grained soil.

Article 1 . . .

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INTRODUCTION

Two electronic piezometers were installed in the ground beneath a building site before construction commenced. These instruments were read frequently throughout the construction period, and then, intermittently for another ten years. The groundwater pressure data collected from this program made it possible to combine the principles of hydrodynamics with Soil Mechanics theory such as to suggest a new model for the use of Geotechnical Engineering. This approach is referred to hereunder as the h-Method (where "h-" stands for hydrodynamic).

What is novel about this project is that it was founded on ~ 36m of normally consolidated, inorganic clays of high plasticity: and this task was to be accomplished without surcharge pre-loading.

PROJECT DEFINITION

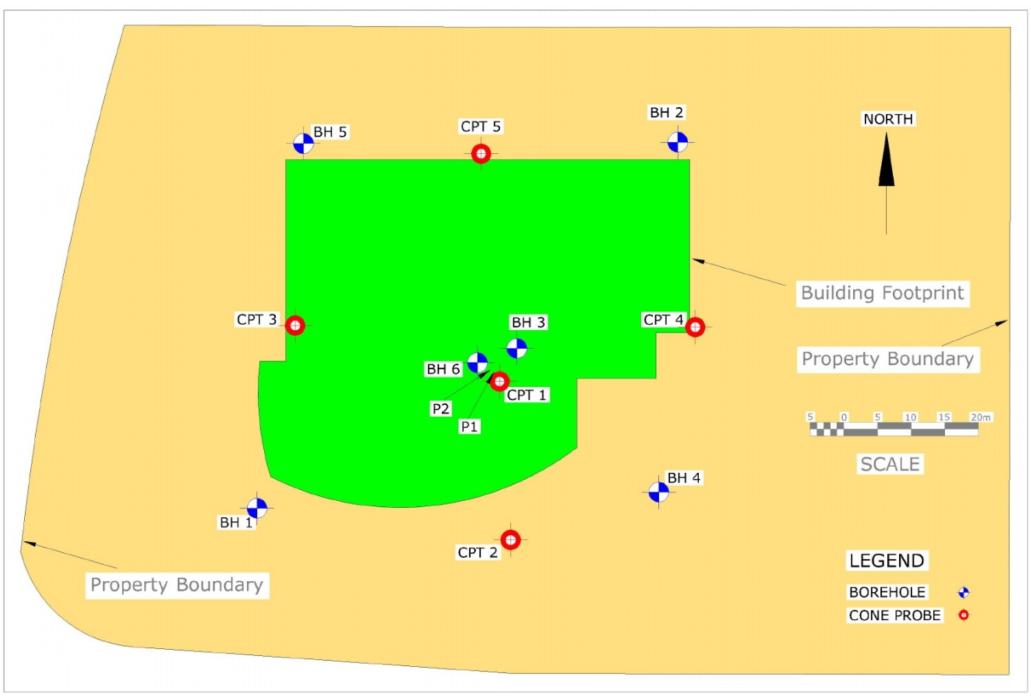

Figure 1 Site Investigation Plan

The site is located in Vernon, British Columbia at the northeast corner where 48th Avenue intersects Highway 97.

The surficial natural ground in this area is too weak and compressible to permit a high profile commercial structure to be erected without prior ground improvement work. Typically, in this neighbourhood, it has been the practice to treat the building area by surcharge pre-loading. In this case those precedents were set aside in favour of a more direct means of foundation improvement, and that entailed removing the surficial deleterious soils and replacing them with non-compressible granular materials of minimal unit weight.

The deeper normally consolidated highly plastic clays were believed to be un-treatable in practical terms by any available technique and were therefore simply left alone. Since this construction method constituted a departure from the local state-of-practice it was considered appropriate to install instrumentation to be monitored during construction, and which would indicate if this non-standard technique was performing according to expectations.

SITE INVESTIGATION

Figure 1 is a plan showing the extent of the site soil investigation work undertaken at this location. It consisted of six hollow-stem auger boreholes (BH1 to 6), five cone penetration probes (CPT1 to 5) measuring dynamic pore pressures, and nine shallow test pits (not shown here). Also, at this time two electronic piezometers (P1 and P2) were installed.

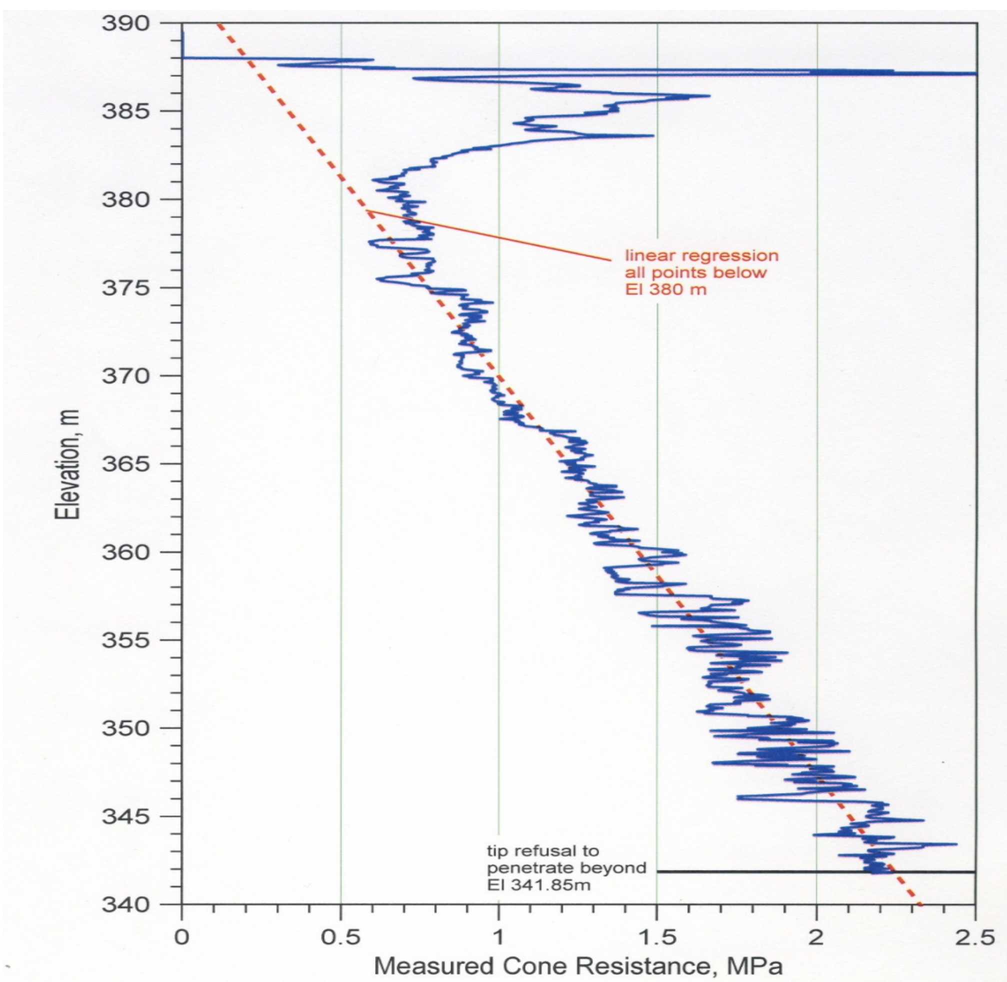

Figure 2   CPT 1: Trace of Tip Resistance

The plot shown in Figure 2 is a trace of the resistance encountered by CPT1 as this cone penetration probe was forced into the ground. Directly beneath the surficial weak and compressible materials a layer of stiff to very stiff clay was found to extend down to about 381.5m. From there, to the point where it met refusal at El 341.8m, the cone trace infers a clay exhibiting multiple interlayering with somewhat coarser grained soils. It is assumed that refusal was due to reaching the glacial till.

A regression analysis performed on all the data below El 380.0m gave a good linear fit; and since this line intercepted the vertical axis just 3.1m above ground level it seems reasonable to refer to this deposit as being normally consolidated. Likewise, for the sake of brevity the upper stiff to very stiff clay layers will be referred to as the "crust".

The results of the 12 Atterberg Limit tests from undisturbed sampling in BH3 and BH6 are shown in Figure 3 plotted on the standard Plasticity Chart devised by Professor A. Casagrande. All but one of these points fall well into the zone of "inorganic clays of high plasticity". The fact that these data pairs are sensibly parallel with the Line "A" indicating these clays have a common geological origin.

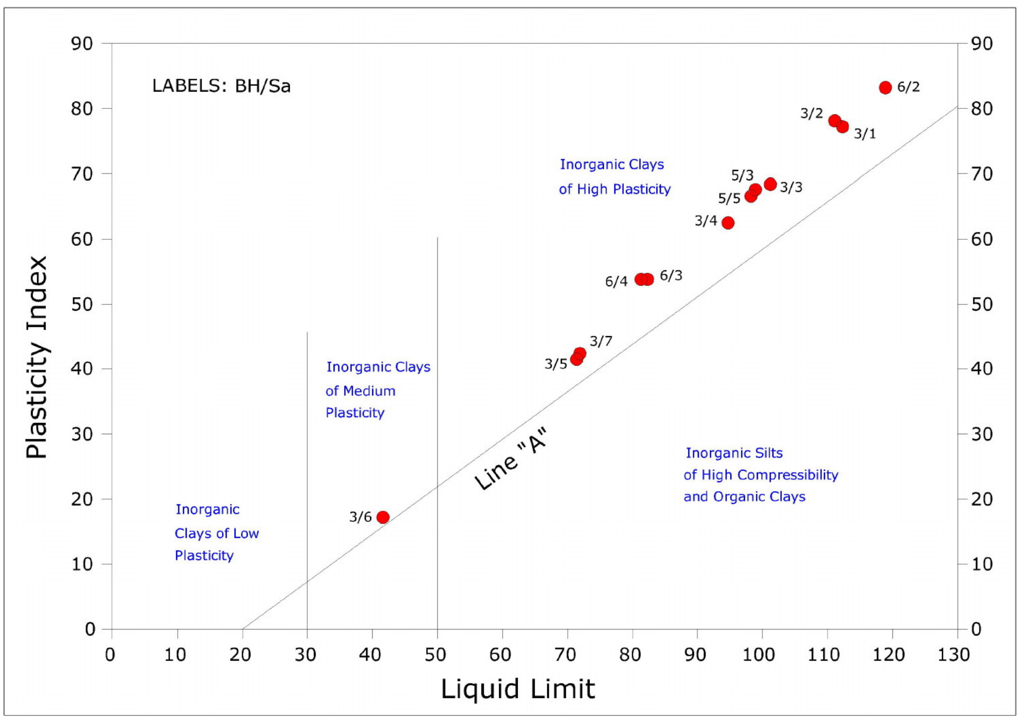

Figure 3 Plasticity Chart

The gradation analysis (hydrometer test) performed on sample 6/2 showed the following composition: 0.2% fine sand, 10.6% silt and 89.2% clay sizes.

Cathode Exchange Capacity testing was performed on three samples recovered from BH6 giving similar results. For example the specimen taken from 6.04m depth showed complete Base Saturation where the individual cathode contributions were: Calcium (Ca^{++}) 78.7%, Potassium (K^{+}) 1.9%, Magnesium (Mg^{++}) 15.6%, and Sodium (Na^{+}) 3.8%.

The laboratory results from the Oedometer testing of 3 specimens recovered from BH6 are plotted on the chart shown in Figure 4.

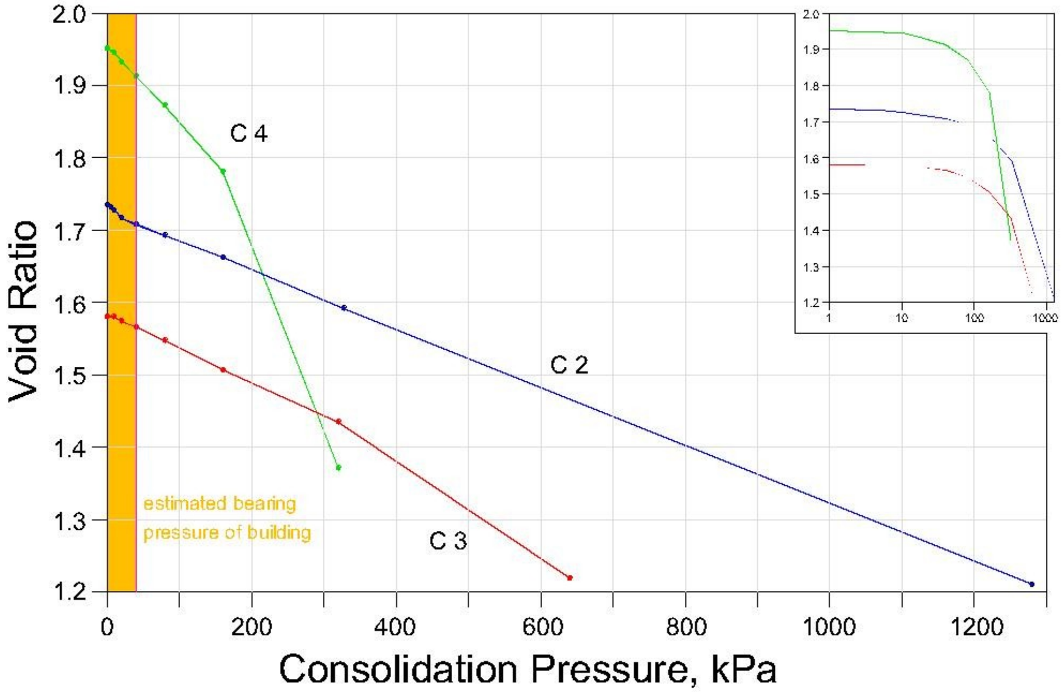

Figure 4 Oedometer test results

The readings suggest that soil sample C2, recovered from the upper crust shows evidence of having been virtually lithified after being baked in the sun to an extent that all pore water evaporated from the particles; a process which brought high interparticle suctions into play. As explained in Ref 1 such forces pull individual particles closer together and result in soil cohesion. In the case of coarse silt and fine sand this type of cohesion (apparent cohesion, C_A) is lost if the soil is re-wetted or becomes totally dry. However, in the case of plastic clay, as drying progresses to the point where *adsorbed* water is lost - thereby allowing physiochemical contact between adjacent clay surfaces - this cohesive bonding will remain should the soil later become (water-) saturated. This latter form of cohesion, is what accounts for the stiffness (terracotta-like) of this specimen.

C3, from 1.5m deeper into the crust, would have been somewhat insulated from solar energy and consequently experienced a lesser degree of desiccation: it appears to have been pre-consolidation to just over 300kPa. C4, from just below the crust, shows a clear indication of pre-consolidation to about 150kPa. Both these levels of pre-compression are far above the estimated cumulative building pressure (45kPa) subsequently to be exerted on the foundations. This average level of construction loading is shown as the highlighted narrow strip to the extreme left of the chart.

The void ratio data shown in Figure 4 indicates that a chunk of this soil would be more water than mineral particles: C2, 64% water and 36% solids; C3, 61% water and 39% solids; and C4, 66% water and 34% solids

GEOMORPHOLOGY

The Geological Survey of Canada surficial mapping of this area indicates that the site is underlain by what GSC term an alluvium-fan complex comprised of “sand, gravel, silt, and muck and peat”. The mapping further suggests that the alluvium is underlain by glacial drift, a very competent material, which was deposited during the Ice Age.

The site specific field and laboratory work discussed above suggest that the following natural sequence of events were responsible for the ground conditions above the glacial drift: The plastic clay was deposited in a lacustrine setting, in a lake which formed behind an ice-dam some distance downstream. When this dam eventually failed the lake level dropped, thereby exposing the clay surface to direct sunlight. The Oedometer testing, as well as its brown colour (oxidation), testify to subsequent desiccation of the upper few metres of the clay.

It seems that the zone immediately beneath the crust, El 377.5m to El 381.5m, has been stiffened somewhat by its contact with the underside of the crust. This can be explained by the equilibration of the moisture contents between the dried-out-clay and the very wet normally consolidated clay underlying it. The suction attending capillary attraction would have drawn water out of the voids of the wet soil and into the empty voids of the desiccated soil; consequently, the strength of both components would have been altered accordingly.

Figure 5 Postglacial Deposits beneath Site Fill

The interlayered weak and compressible materials overlying the crust, which are somewhat bluish in colour (not exposed to the atmosphere), were subsequently deposited in periglacial conditions. These soil types are similar to those common in the Canadian North. The presence of volcanic ash layers within this stratum helps fix the time of deposition at sometime following the eruption of Mount Mazama which occurred about 7,700 year ago. This ash was blown around in the Vernon area for up to 3,000 years after the eruption, and this accounts for more than one seam of ash being present at the site.

The pair of piezometers which are discussed later in some detail, show that an artesian condition exists within the plastic clay stratum. This is consistent with the proximity of the valley wall to the west.

ENGINEERING CHARACTERISTICS OF GROUND

There are basically four distinct strata beneath this site: The post-glacial natural surficial materials shown in Figure 5 underlying the old site grade fill, can be seen to contain multiple layers of weak and compressible materials including very soft clay, peat, ash, and shells, and as such must be ruled out as a possible foundation layer on which a building might be supported. Likewise, at the other extreme, the highly competent glacial drift body at a distance of 36m (118ft) below ground level is simply out of reach in practical terms. That left only the desiccated "crust" and the underlying normally consolidated lacustrine deposits as the foundation options.

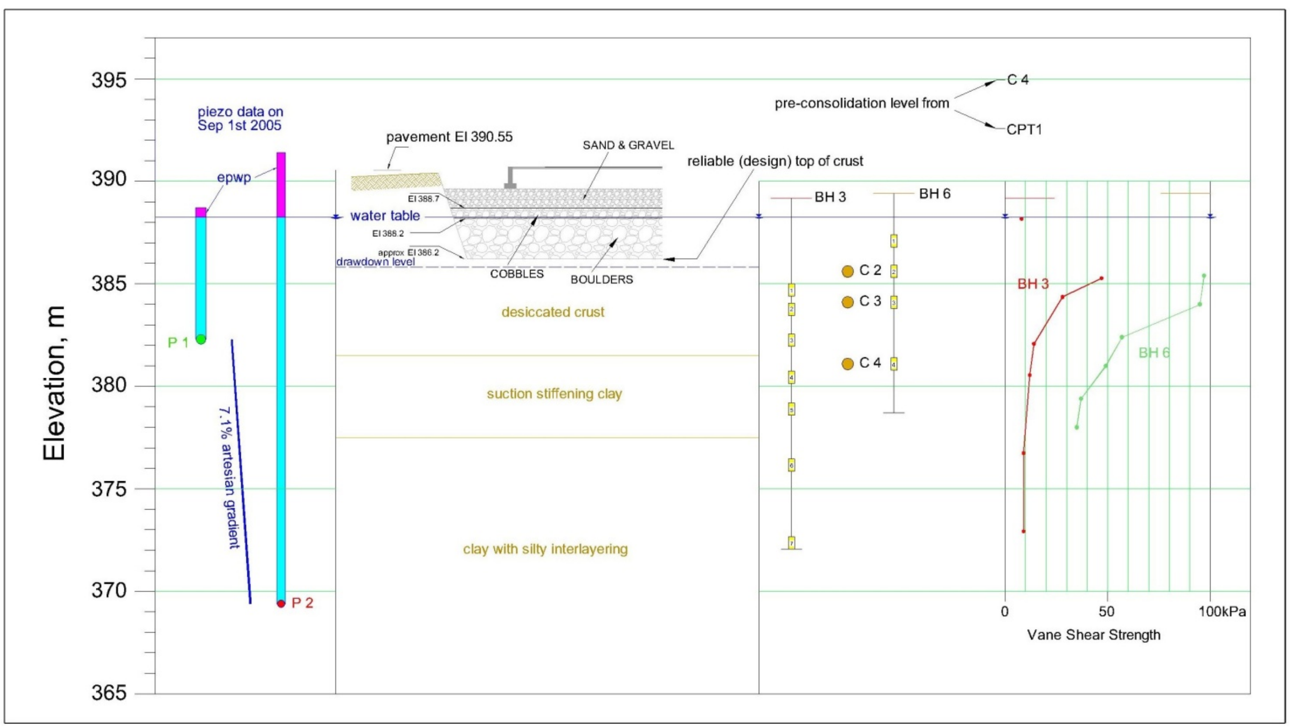

Figure 6 Foundation Design shown against S.I. Findings

Summary of Field & Laboratory Data

Figure 6 is a sketch drawn for the area shown in Figure 1 where BH3, BH6, CPT1, and the two piezometers P1 and P2 are tightly bunched together. What may be seen from left to right are :

- The metric geodetic elevation scale against which the data is aligned.

- The depths at which the piezometer tips are set and the artesian groundwater condition which pertained between them before construction commenced; plus the data which was recorded at a particular date in order to illustrate the term ***epwp*** which is explained below**.**

- A cross-section through the structural foundation elements and the underlying natural strata as determined by Site Investigations [S.I.].

- The depths at which undisturbed samples were taken and where the consolidation specimens were selected.

- The pre-consolidation pressures suggested by the cone penetration test CPT1 and laboratory consolidation test C4.

- The field vane shear test values from boreholes BH3 and BH6.

It is important to note from the geometry depicted in Figure 6, that while the initial working surface was at elevation 389.3m the water table is at 388.2m. This establishes that the level at which the building loads are being placed in the dry is more than a metre above the water table. Since the materials in between are coarse granular fills there cannot be any moisture continuity between ground surface and the top of the saturated foundation strata below.

Foundation Design

Given the site stratigraphy, our design came down to recommending that the existing materials above the crust surface be excavated and replaced by imported granular fill, and that the desiccated clay layer, which was found to be present and consistent across the site, be designated the bearing layer for the structural foundations.

The selection of backfill to replace the surficial postglacial mixture of weak heterogeneous materials was designed to minimize the unit weight to be imposed on the foundation. Towards that end a fill comprised of poorly graded (uniform grain size) particles of boulder size, and that overlain by cobble size of similar gradation was chosen: the stipulation being made that the size of cobbles would be such that this layer of smaller particles would not fall into the boulder voids. An additional constraint on the cobble gradation was that normal sand

and gravel fill, would likewise, not enter the cobble voids. Another benefit of using such large size particles is that such could be placed through standing water without concerns about segregation. The idea throughout being that the resulting fill would have high void ratio, and consequently, low bulk density.

Both the bottom layer of boulders and the intermediate layer of cobbles were selected and approved by prior inspection at the quarry.

Figure 7 Backfilling the Foundation Excavation

Figure 7 is a photograph taken as the foundation excavation was being backfilled. The body of the picture shows the boulder layer almost complete. The top of the "crust" is still exposed at bottom right, beside the cut slope. On the far side of the red hose pipe, stockpiles of the cobble fill material may be seen. And, further back, beyond the boundary chain-link fence, the pre-load fills intended to prepare the foundations for what was shortly to become the Lake City Casino are obvious.

GROUNDWATER PRESSURE MONITORING & INTERPRETATION

What follows is an interpretation of the piezometric pressure readings measured in the groundwater during the course of the various activities which attended ground preparation and building construction at this site.

Installation Details

Two electronic piezometers (Adara Systems Ltd) were implanted in the natural ground at the location shown in Figure 1. Piezometer #1 [**P1**] was set at Elevation 382.3m and Piezometer #2 [**P2**] at Elevation 369.4m, that is, at 7.0m and 19.9m below ground level, respectively. It can be seen in Figure 5 where they reside with respect to the foundation stratigraphic units. These instruments showed right away that both were resided within an artesian condition which was producing upward flow at a gradient of 7%. They were subsequently monitored for another 10 weeks before construction work began. This period allowed the pore water pressure escalation, caused by the cavity expansion straining resulting from their having been pushed into the ground during installation, to dissipate. Then, after construction work started in October 2005, readings were taken on an intermittent basis throughout the following 12 months - a duration which covered the full construction period.

The h-Method

This rather extensive record showing how the groundwater reacted/responded to having a commercial structure built atop it will be analysed using a procedure referred to as the h-Method (hydrodynamic - method). The h-Method is a convenience devised whereby some of the principles of Fluid Mechanics have been called upon in the aid of Soil Mechanics in order to formalize how Geotechnical Engineering ought to treat the relative movement of the two phases (solids and water) within a water-saturated soil.

The key measure used in this method is "excess pore water pressure" [***epwp***]. By definition, *epwp* is the piezometric pressure measured at the tip of a piezometer **minus** the hydrostatic pressure appropriate at this same depth. It is therefore (taken to be) the potential energy available to do work from this same point/location. The water table is zero potential and epwp cannot go negative.

The h-Method takes the following principles/rules as being axiomatic:

- In water-saturated soil the two phases inhabit the same space, but they do not affect one another unless/until one phase moves with respect to the other.

- The relative movement between the solid-phase and liquid-phase in a two-phase system is the cause/source of excess pore water pressure (*epwp*).

- If there is no relative motion between the water and the soil-structure, then the pore pressure at all points will be hydrostatic.

- When *epwp* exists, so does a hydraulic gradient, and in consequence seepage flow will be taking place somewhere within the system.

The prime source of *epwp* is believed to be the consolidation of saturated ground under the stress of increasing overhead loading. The classical Terzaghi equation is not appropriate here since its use is restricted to vertical drainage in a continuum, whereas what is needed here is a solution which can deal with radial flow through a discontinuous soil-structure (whose volumetric makeup is 60% fluid).

Consolidation

The h-Method holds the following mechanistic view of consolidation:

When an increment of load is added to the surface of a foundation which is not strong enough to carry it, then in order to maintain vertical equilibrium, that new total pressure, must be carried down to a depth where it encounters a denser material which has the competence to bear it. Consequently, the soil column between ground level and its bearing layer is put into compression; thus, consolidation is initiated.

This process involves the expulsion of water from the intervening soil-structure so as to allow the solid phase to form into a denser agglomeration of particles. Such a concentration of the solid phase material process results in a stronger aggregation because it makes for more interparticle mineral contacts.

In this integrated system it is appropriate/necessary to account for the effect that the downward movement of the solids has on the liquid-phase. The lowering of the centre of gravity of the solids produces energy (weight times distance, $F * L$) in the system which can best be accounted for by pore water pressure increase (pressure times volume, $F/L^2 * L^3$) in the liquid phase. So, it is contended from this mechanistic understanding of consolidation that the source of energy to support/sustain the exit gradient (*epwp*) comes from the gravitational potential shed by the descending solid particles.

It is only the liquid phase that can express itself outside the geometric confines of the consolidating column. As a water pressure wave, it is propagated throughout the regional waterbody and can be felt throughout that domain immediately and without diminishment.

Interpretation of a Continuous Data String

Figure 8 shows the water pressure history recorded by P1 at 6 minute intervals over a period of seven days. A direct digital comparison of the simultaneous readings produced at P2 over the same week showed that the pore pressures measured by the two piezometers, 12.9m apart vertically, were identical in amplitude, duration, and timing.

Such instantaneous and undampened transmission is evidence that the physical medium through which the pore water is effected is a 2-phase system, just solids and water. If the medium had been 3-phase (solid, liquid and vapour) the entrained air would have interfered with energy transmission. This is a conclusion which could not have been drawn with certainty from the geomorphology, since although these clays are ancient natural deposits of inorganic materials deposited underwater, there was a period during which the crust (in which P1 is embedded) was exposed to the atmosphere and baked dry.

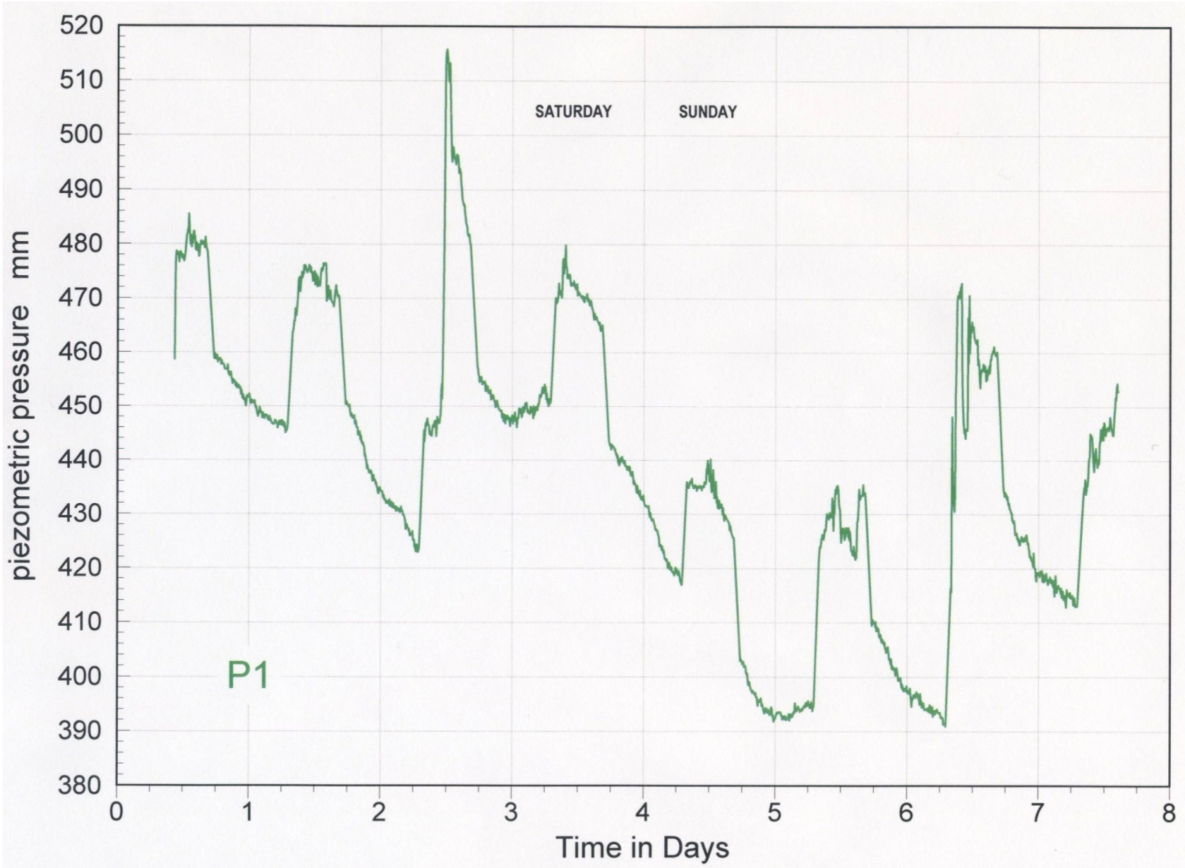

Figure 8 Extraneous Ground Water Pressure Pulses

These data points, which incidentally are *epwp* values, are the combined water pressures emanating from two distinct sources: those resulting from the increasing construction loads being added above ground level; and a pressure pulse of constant intensity (22mm/ 0.9" of head) entering the groundwater every morning at 7am and leaving at 5pm that same afternoon. Obviously, pressures from different sources are simply commensurable – there are no different flavours associated with how a pressure came into being.

Although the external source of this interference was not definitively tracked down, it is likely that a municipal lift-station situated some 200m northeast of the site was the cause. A leak in the output side of the plumbing would account for the pulse.

The pulses are cleanest on the weekend when activity at ground level was minimal. The pressure spikes superimposed on the repetitive pulses during working days are attributable to the ever-increasing ground level loading, due for example, to concrete placement and building material arrivals.

What is particular interest is the fact that during the quiet interval between the Saturday and Sunday pulses a gradual dissipation of water pressure can be clearly discerned. Incidentally, this observation confirms the axiom that when/where *epwp* exists within a saturated soil, seepage flow must also be taking place.

A Necessary Explanation

As shown in Figure 6, the loading zone is hydraulically isolated from the saturated foundations. So the question naturally arises as to how do increments of surface loading in the dry show up in two piezometers at depths of 7.0m and 19.9m below ground level; reading which are plausibly commensurate with building progress?

In terms of the h-Method this situation may be addressed as follows:

It is necessary that the imposition of a surface load find an equal and opposite reactive force somewhere within reach beneath. Therefore, new loading added to the dry ground above the water table must be carried by the soil-structure down the underlying soil column to a level which can sustain it. In so doing, this soil column is put into compression. Beneath the water table the commensurate strain involves, of necessity, relative motion between the solid and water phases. Such interaction between the phases causes water pressure to be generated ahead of the moving solid particles, see Ref 3.

The fact that water pressure shows up at depth below ground level is proof positive of the proposition that pore water pressure is the result of soil-structure straining.

Piezometric Record

The groundwater pressures measured by the two piezometers throughout the construction period from July 2005 to August 2006 are shown plotted in Figure 9.

The trace for the shallower instrument (7m depth) is labelled P1and that of its partner (19.9m deep) is labelled P2. The natural water table is shown as a blue line at Elevation 388.2m; during the two periods of site groundwater drawdown the temporary water table is shown in the bottom left as a purple line. The vertical distance from the prevailing water table to a point on either trace is the numerical value of the *epwp* at that time and that piezometer.

The sixteen Upper Case letters (A to R) shown against the traces are listed in Table 1; these are used as signposts to specific events such as groundwater drawdown. The numerical value listed against each letter is the *epwp* (in metres of water) available for that point at that particular time. Incidentally, part of this *epwp* is attributable to the artesian pressure gradient.

These plots start on July 14th, three days after installation, and show that over the next six or seven weeks the *epwp* due to cavity expansion (a consequence of pushing the piezometer bodies into the ground) dissipates and both readings tend towards their minimum values [A,B].

The next matter of note is that the piezometric elevation of P2 is just about a metre higher than that of its companion P1 at any time. This indicates that an artesian pressure gradient exists in the groundwater between the tips: This is not surprising given the nearby mountains and the layered nature of the foundation soils. Such a condition is beneficial to the project since it comes with a minimum upward Seepage Force of 9kPa (~200psf) in support of the foundation.

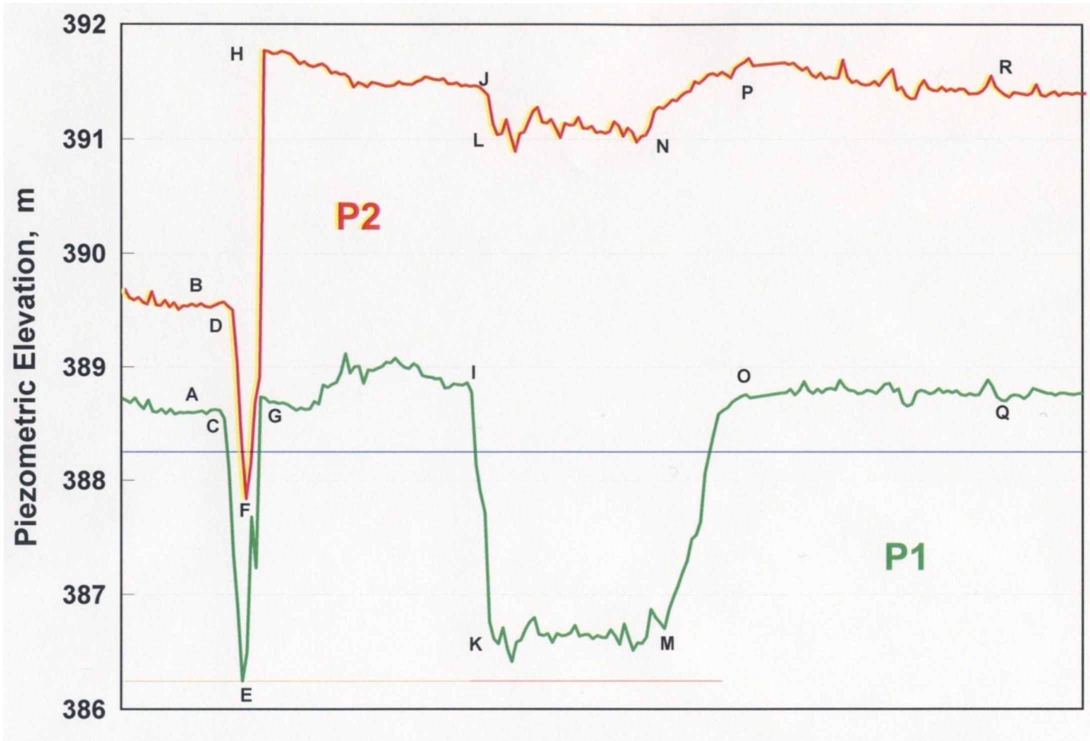

Figure 9 History of Ground Water Responses to Construction

At the beginning of October 2005 [C,D] the first significant scheduled construction activity began: this was the excavation of the weak and compressible natural materials beneath the building footprint in order to remove those postglacial soils and expose the competent desiccated clay crust. In the event, the contractor opted to dewater the excavation because the wet spoil being hauled from the site would have spilled from the trucks and muddied / soiled the highway pavement. By December 1st the hole had been backfilled with boulders and cobbles and the water table allowed to return to its natural elevation.

P1 responded to this as expected, with a 23kPa pore water drop consistent with the amount of burden removed [E], and subsequently returning to an appropriate level [G] as the hole was refilled with granular soils and then flooded again. But the recorded behaviour of P2 was most surprising: between [D] and [F] it dropped by 16kPa, and then, by December 1st rebounded by 39kPa to [H] (an abrupt increase of 3.93m). This was a serious departure from expectations and suggested that following the shock of drawdown the crust proved too weak to fulfil its load-carrying function, and was just passing the increasing construction loading onto the softer sensitive fine grained stratum below.

TABLE 1

Date	label	*epwp*	lable	*epwp*
August 23, 2005	**A**	0.33	**B**	1.25
October 5, 2005	**C**	0.29	**D**	1.28
November 3, 2005	**E**	**0.00**	**F**	1.59
December 1, 2005	**G**	0.48	**H**	3.52
March 3, 2006	**I**	0.52	**J**	3.21
March 17, 2006	**K**	0.17	**L**	4.65
May 29, 2006	**M**	1.82	**N**	5.32
August 1, 2006	**O**	0.40	**P**	3.10
September 1, 2006	**Q**	0.46	**R**	3.14

The consequence was that henceforth, rather than dealing with a near-elastic crust material, the building would be subject to consolidation. As this event was not accompanied by any surface manifestation, and the pore pressure began to drop during the next few days, the incident passed without comment. Nevertheless, as it did illuminate that an overestimation of the crustal strength had been made at the design stage, that computational error will be returned to in the Conclusions section below.

As may be seen in Figure 9, for the next three months until the second drawdown, starting March 3rd, the traces each show a different profile : P1 is jagged [G to I], whereas P2 [H to J] is muted. This is because the former is in the desiccated crust with easy access to drainage, whereas the latter resides within the inorganic clays of high plasticity where venting of *epwp* is dependent on consolidation.

The P1 trace is more irregular than that of P2. This is because P1 is embedded in a more elastic soil than its partner which is experiencing reconsolidation. So P1, during this period, shows the net *epwp* caused by loading added to the ground surface, minus its concurrent dissipation by seepage flow. The leading edge of each peak represents a situation when surface loading exceeds the rate of drainage; the trailing slope indicating where pore pressure dissipation is faster than the effects of ongoing construction.

At the beginning of March [I,J] the local water table was drawn down for a second time, on this occasion to enable installation of underground facilities "in the dry". The response of P1 was as anticipated: the readings during this period [K, M] all fell by the amount of the drawdown (2.0m) in acknowledgement of the new local water table, but while still retaining the proper levels of *epwp*, about 0.3m. As for P2, it did not follow the example of P1 inasmuch as to recognize the full extent of the water table drop.

The rational advanced for this departure from the rule is as follows: It is argued/surmised that because the water pumped from the foundation was not taken away from the site, but rather discharged into the highway ditch just beside the west property line, it may be presumed that it re-entered the local groundwater, with the consequence that the lower piezometer P2 did not have the opportunity for its pore pressure readings to fully reconciled with what was going on 12.9 metres above.

Comparing the shapes of the two sets of readings during the following three month period, while both instruments are obviously responding to the same stimuli, their treatment of those pressure changes was somewhat different. For the first half of this time [L-N] P2 shows more pronounced peaks and valleys than its partner. The reason for this seems to be that in the beginning P2 records the full extent of the changing *epwp*, but with time, this intensity becomes subsumed beneath its ongoing background consolidation. The response of P1 [K-M], on the other hand, seems to be a more natural matter of gaining some load while at the same time losing some *epwp* to the effort of drainage.

At the end of May [M,N] pumping ceased and the natural water table restored itself by the beginning of August [O,P]. Thereafter, to end of construction in September [Q,R], the foregoing relationship between P2 and P1 appears to fit the data well, with the peaks being more prominent in P2 while consolidation continues at a rate suggesting it would take a matter of years to exhaust its remaining pore pressure potential. P1 just drifts on at no more than 30cm above its artesian level.

Post-construction Readings

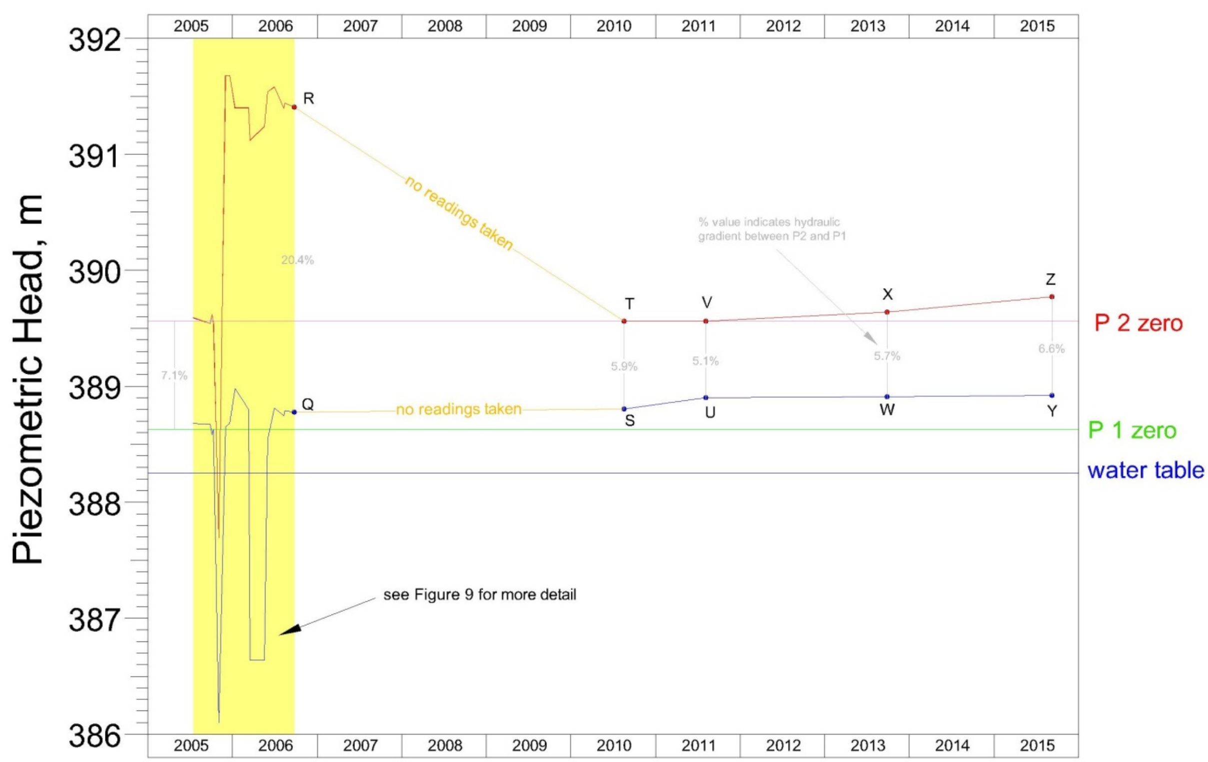

Figure 10   History of Piezometer Responses from 2005 to 2015

For the 3½ years following the end of construction no further piezometer readings were taken. Then starting in late summer 2010 a series of four single pairs of data points were read over the next five years. Those readings are shown on Figure 10 together with the water table level and the artesian elevation for the two piezometers.

This plot shows that in the meanwhile the consolidation of ground around P2 had fully resolved itself and the pressure reading [T] was at its minimum artesian level. But this was not the case with P1 where the pressure at [S] was 0.17m above its artesian zero. And looking back to Figure 9 it can be seen that P1 had not lost any potential in the 3½ years since [Q].

The h-Method explanation of this apparent anomaly is as follows:
Back in November 2005 at the time when the crust failed (under the strain reversal of the first drawdown) the hydraulic continuity of the groundwater flow from P1 to the water table was damaged/impaired to the extent that it required the expenditure of more epwp to maintain the flow through the upper crustal soil. Consequently, the pressure around P1 needed to be permanently increased by 1.7kPa, thereby reducing the original artesian pressure gradient from 7.1% to 5.9%: a reduction to the original beneficial upward Seepage Force of 16.7%.

P1's subsequent increase [S-U] is attributable to the building of a Casino on the neighbouring property immediately to the north of this site. P2's escalating readings [Y-X-Z] are probably due to further development of the area at a somewhat more remote proximity. What these few readings suggests is that, just like the off-site pulses discussed earlier (see Figure 8), these pressures were transmitted as *epwp* through the water-phase, and had their origin in relative motion in the foundation strata beneath the nearby buildings.

CONCLUSIONS

Structural Performance

Little needs to be said in this regard other than to draw attention to the evidence offered in Figure 12. This photograph, taken after 10 years of commercial operation, shows the only crack, about a metre long, in the entire 7,000 square metres of concrete flooring. Incidentally, that single imperfection was caused by a machine bumping into the column from which it emanates while the concrete was yet green.

From a practical/implementation point of view the project was successful and the building contractor had no particular difficulty in implementing the more novel aspects of the foundation design.

It also seems fair to say that the measured behaviour of this novel foundation design proves that the hydrodynamic approach (h-Method) used here is valid, and furthermore, demonstrates the diagnostic and explanatory power of this approach to geotechnical engineering.

Figure 11 Only Floor Crack in Entire Concrete Slab

Questionable Decision in Assessing Strength of Crust

The purpose of this post-mortem is to decide, if a mistake was made at the design stage, how best to learn from it and how to avoid it in future.

During November 2005 the P2 record suddenly, and without any apparent reason jumped 3.9m (38.6kPa). This was a cricis point inasmuch as what the instrumention indicated was that the crust was no longer supporting the building loads, instead these were now being carried directly into the normally consolidated softer clay stratum below. Fortunately it took only a matter of days to see that the P2 readings did not show any further degradation in this situation, and in fact had stabilized and begun to come down, indicating that things were getting better with time.

The strength data from the six boreholes and five cone penetration probes put down during the site investigation showed (*with only a single exception*) a consistent picture of a 4m thick, stiff clay layer, at shallow depth. This was designated the foundation bearing layer at the design stage. The exception was one of the two strings of borehole vane shear testing shown to the right of Figure 6. That of BH3 showing far weaker strength readings than BH6. ConeTec Investigations Ltd, a highly regards geotechnical investigation contractor, produced the BH6 data string, as well as the CPT probes; all of their work was internally consistent. The only red flag, BH3, was data provided by a good local driller. In the event, believing that both sets of information could not be correct, the design adopted BH6 and set the BH3 data aside on the supposition that their vane calibration was questionable. This was a mistake: the proper course of action would have been to have asked the local drilling contractor to have their vane's calibration verified by an independent laboratory.

It should be acknowledged that it is possible both vane tests were right, each produced values appropriate to the soil type surrounding them: the higher values reflecting intact soil; whereas lower strengths might well have come from weaker soil adjacent to fissures. The fact is that the existence of hexagonal fissures typical of sun-baked clayey deposits was not considered, the reason being that the entire surface of this crust was exposed during the foundation preparation and there was nothing of this sort to be seen. And given the periglacial environment in which these clays are deemed to have been deposited the surface evidence was accepted.

Notwithstanding the forgoing arguments, it will be shown in the third of this current series of articles, *Compacted Earthfillis a 3-phase Material*, that because of the asymmetric nature of the "wetting-drying cycle", once a fissure opens in a fine soil such as this, it requires very high ambient water pressures to close it again. In addition it should have been realized that the machine blade which excavated the overburnen, in smearing the moist surace, would have obliterated any evidence that might have been there.

What is to be learned from this experience is that the likelihood of hexagonal cracking in desiccate clays needs to be kept in mind during design.

Foundation Flow Nets

Since *epwp* is a simple, and complete, accounting of the unspent/available water potential at the tip of each instrument, it would be an easy task to draw a flow net for the ground water beneath a construction site. Even in the minimal case of this site, with just three separate points along a vertical line, it would have brought out the fact that the conductivity of the upper crust had been reduced following the dewatering effort.

However, in the case of a large project which could carry the cost of many piezometers spread out laterally and vertically across the ground beneath a site a 3-dimensional flow net could be drawn to monitor what was happening within that submerged volume. Once the spatial relationship of each instrument tip to the water table was established it would then be just a matter of drawing contours of energy magnitude (*epwp)* within this field. Those contours would be predominantly horizontal surfaces which would show a vertical drop in potential as the ground water level (zero potential) was approached. Then, in order to complete the net, flow streamlines could be drawn orthogonally to the equipotentials. Such a graphic would reflect the *epwp* response to surface loading overhead; a matter of some importance to buried structures such as tunnels.

The overriding rule is that once an external load is applied to the surface it causes an appropriate response in the groundwater which is recorded by any piezometer within the field. This is instantaneous, and involves an immediate hydraulic gradient being established between other points in the system. Seepage will then take place in proportion to the gradient and the permeability (v=i k) between pressure points until eventually, after outside loading ceased, all *epwp* will be exhausted and the groundwater return to its preconstruction state, that is, hydrostatic or natural artesian.

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