Pore Pressure Monitoring of a Novel Foundation

Written and illustrated by: William E Hodge

Table of Contents

List of Tables

List of Figures

Attachment

1. INTRODUCTION

There are three things about this particular project which help to lift it out of the ordinary and make it worthwhile writing up and passing along.

The first and most important was that the Owner was an individual, rather than a group, and a young intelligent man who was prepared to invest in novelty so as to get going with building his new business venture with the least delay.

Secondly, the foundation stratum was underlain by ~ 39m of normally consolidated clay, whereas the superstructure was rather heavy and brittle, and this was to be accomplished without pre-loading.

Thirdly, two sensitive electronic piezometers were installed in the clay which were used to monitor the response of the foundation stratum to building loads, and which have since been read intermittently for over ten years.

Interpretation of piezometric data subsequently established the validity of the hydrodynamic approach (h-method) to geotechnical engineering practice.

2. PROJECT DEFINITION

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

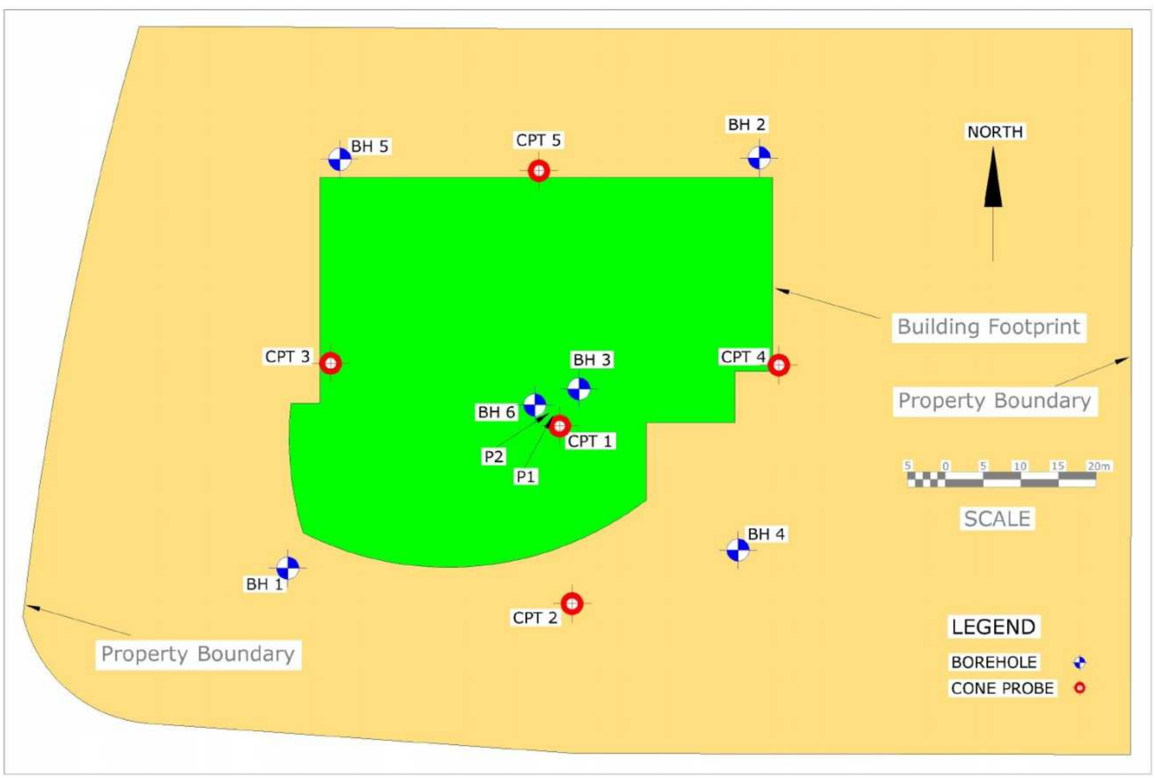

Figure 1 Site Investigation Plan

The 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.

3. 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 5), five cone penetration probes (CPT1 to 5) measuring dynamic pore pressures, and nine shallow test pits (not shown here). At this time two electronic piezometers (P1 and P2) were installed.

Figure 2 Interlayered very soft clay, peat, ash, and shells (~1m visible) beneath imported fill

Figure 2 is a photograph of the natural soils underlying the old site grade fill and shows this stratum to contain multiple layer of weak and compressible materials including very soft clay, peat, ash, and shells, such as to rule it out as a possible foundation layer on which a building might be supported.

4. ENGINEERING CHARACTERISTICS OF GROUND

The plot shown in Figure 3 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, down 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 detailed interpretation of the full digital output of this push is giving in the Attachment.

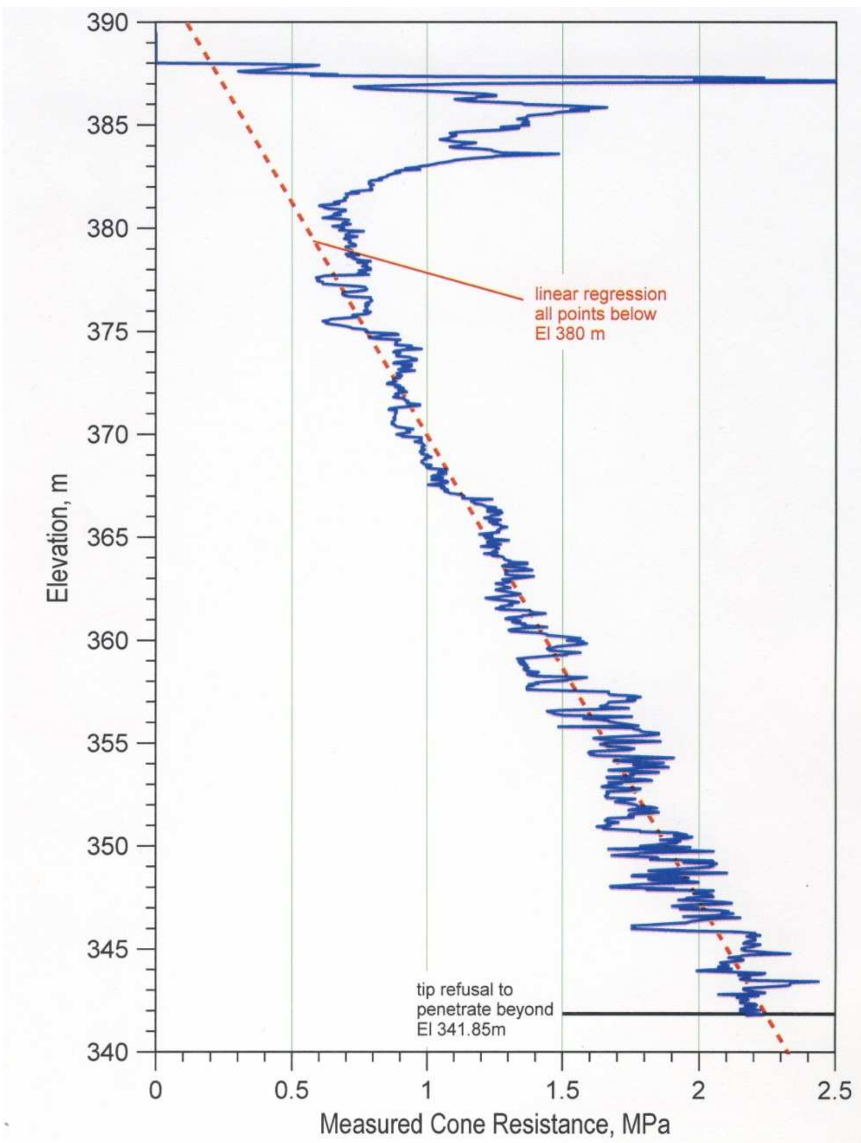

Figure 3 CPT1 tip resistance

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 is reasonable to refer to this deposit as being normally consolidated, rather than lightly over consolidated. For the sake of brevity I will refer to the upper stiff to very stiff the clay layer as the "crust", and the deep interlayered clay above the till as being normally consolidated.

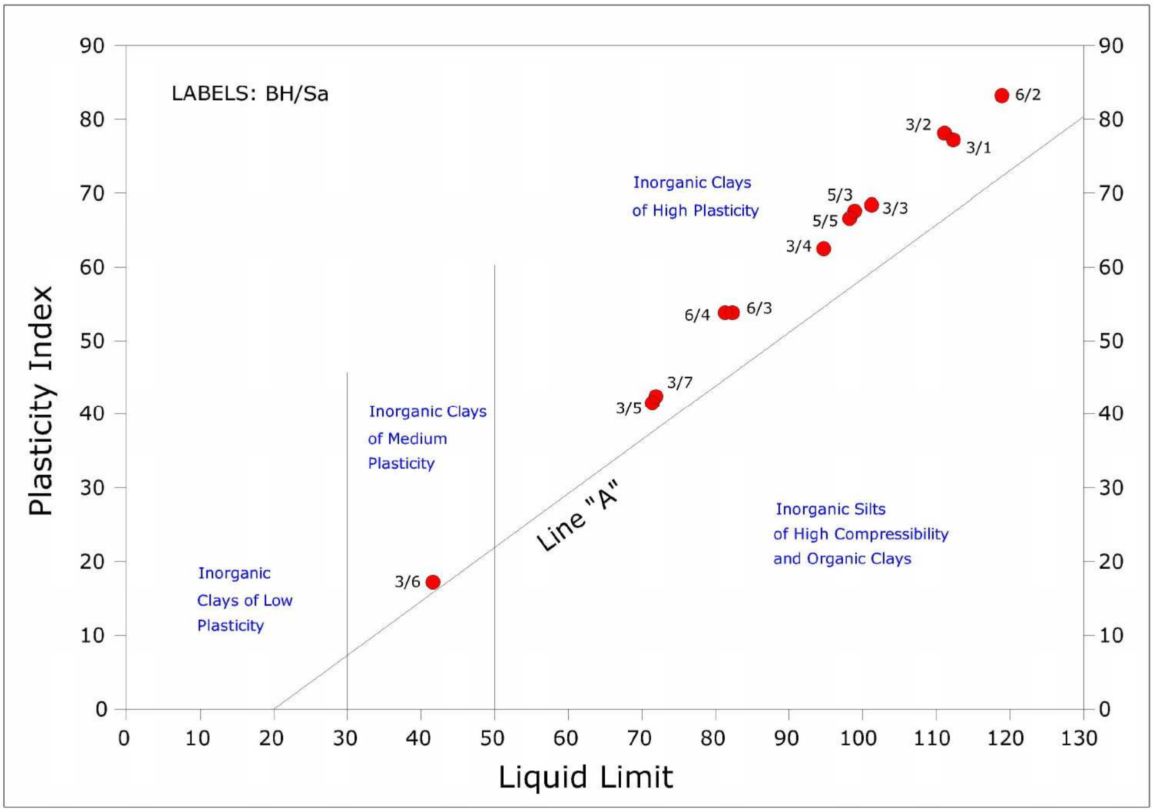

Figure 4 Plasticity chart

The results of the 12 Atterberg Limit tests from undisturbed sampling in BH3 and BH6 are shown in Figure 4 plotted on the standard Plasticity chart devised by 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 5 is a sketch drawn for the area in which BH3, BH6, CPT1, and the two piezometers P1 and P2 are all bunched. It shows from left to right:

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

b. The depths at which the piezometer tips are set and the artesian groundwater condition which pertained between them before construction commenced.

c. A cross-section through the structural foundation elements and the underlying natural strata.

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

e. The field vane shear test values from BH6

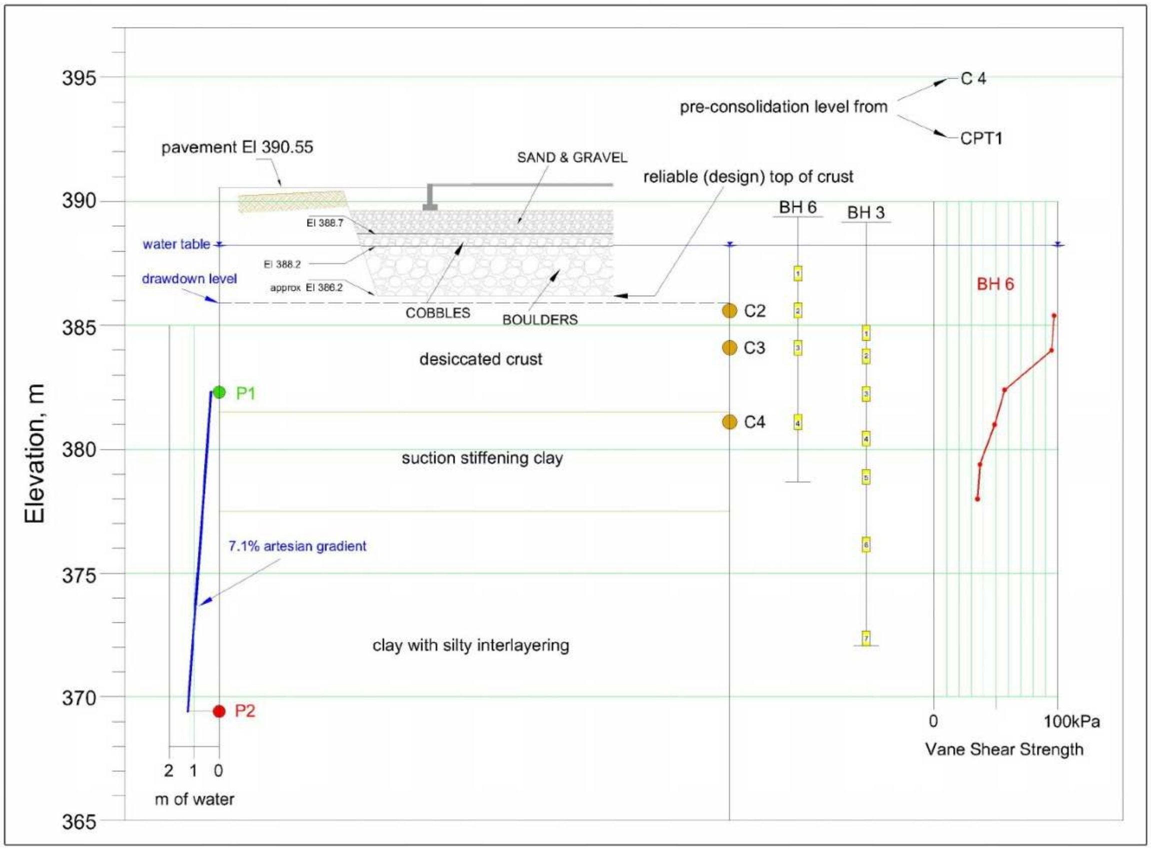

Figure 5 Summary of field & laboratory findings

The laboratory results from the Oedometer testing of 3 specimens recovered from BH6 are plotted on the chart shown in Figure 6.

C2, recovered from the upper crust shows evidence of having been virtually baked in the sun to an extent that all pore water evaporated from the particles, a process which brought high interparticle suctions to come 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 (C_A) is lost if the soil is re-wetted, or becomes totally dry. But 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. And, this latter form of cohesion, is what accounts for the stiffness (terracotta-like) of this specimen.

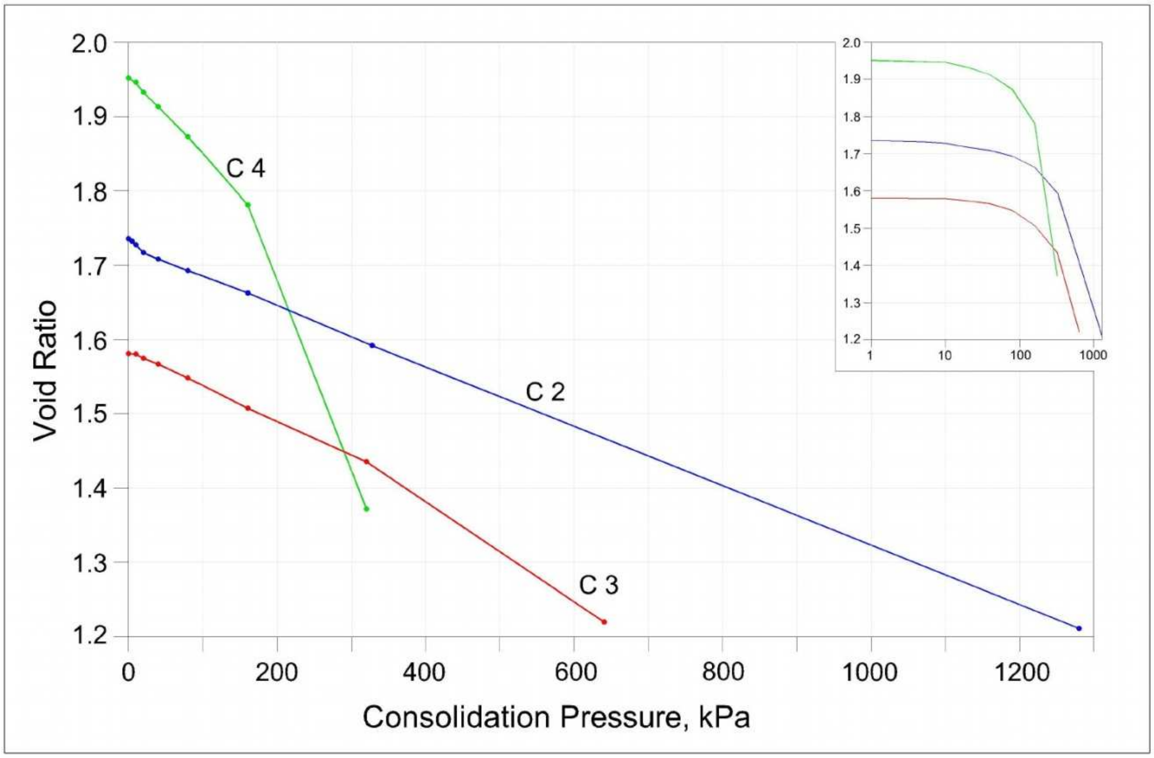

Figure 6 Oedometer test results

C3, from deeper in the crust, would have been somewhat insulated from solar energy and consequently experienced a lesser degree of desiccation. There are insufficient Oedometer points to be sure of its pre-consolidation pressure.

C4, from just below the crust, shows a clear indication of pre-consolidation to about 150kPa. This suggests the top level of the clay deposit was at one time El 395.0m, an estimate which compares reasonably well with the CPT1 indication of El 392.6m.

5. 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 which is locally referred to as "till". The till was deposited by glacial action during the Ice Age. The site specific field and laboratory work discussed above suggest the following natural sequence of events were responsible for the ground conditions above the till:

The plastic clay was deposited in a lacustrine setting, in a lake which formed behind an ice-dam some distance downstream. When this dam melted the lake level dropped thereby exposing the clay surface to direct sunlight. The oedometer testing as well as its brown colour (oxidation) testify to the subsequent desiccation of the upper few metres of the clay.

It seems to me that the zone immediately beneath the crust, between El 377.5m and El 381.5m, has been stiffened somewhat by its contact with the underside of the crust. The mechanism to which I attribute this local enhancement of cohesion is suctional forces from the overlying dried-out clay reducing the void spaces in this zone and thereby produce a more resistant/stronger soil-structure: The suction would be a result of the capillary attraction of water towards the desiccated void spaces.

The interlayered weak and compressible materials overlying the crust, which are predominantly bluish colour (not exposed to the atmosphere), were subsequently deposited in conditions which are called periglacial, that is, close to a glacier. 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.

6. GROUND PREPARATION FOR BUILDING

The surficial ground conditions underlying this site, as already explained, consist of weak and compressible soils, and if a structure had been built here using shallow footings, without first improving the ground in some significant way, the structure would have been vulnerable to unacceptable deformations.

Piles were not a viable solution for obvious reasons. And the option of improving the ground by the technique of surcharge pre-loading did not appeal to me because the presence of peat in the surficial natural stratum raised the issue of secondary consolidation over the long-term. Also, there was the possibility that the existing fill incorporated random degradable &/or deleterious materials which could in time deteriorate, especially if pushed below the water table as the underlying materials compressed.

For these reasons I recommended 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 made the bearing layer for the structural foundations.

The selection of backfill material was key to this approach: By using a fill comprised of poorly graded (uniform grain size) particles of boulder and cobble size, the resulting fill would have the following favourable characteristics:

a. A uniformly sized aggregation of coarse sub-rounded particles would not segregate as it passed through standing water and the resulting accumulation would achieve an acceptable level of density without the need for further compaction effort.

b. Such materials sizes are commonly produced at a quarry as the discard from processing sand and gravel, it being the oversize separated by passing over the "grizzly". In some areas it is considered a waste by-product, and consequently, relatively inexpensive.

c. But the primary reason for this specification was that such poorly graded fills, when placed in separate layers, could be expected to have a void ratio of about 0.35, thereby having a unit weight of only 19.2kN/m^3 as compared with a well graded sand and gravel with a void ratio of about 0.2 with a unit weight of 21.7kN/m^3. Altogether this makes for a small but important comparative load reduction of 4.3kPa.

Figure 7   Backfilling the Foundation Excavation

The bottom layer of boulders was selected and approved by my inspection at the quarry. After this layer was in place the overtopping cobble material was selected in the same way, on the basis that the cobble sizes were such that they would not fall into the boulder voids. The additional constraint on the cobble gradation was that normal sand and gravel fill would likewise not enter the cobble voids.

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 Vernon ("Casino") are obvious.

On this basis the first construction activity, beginning in early October 2005, was the excavation of the weak and compressible natural materials beneath the building footprint in order to 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 the highway pavement. By December 1st the hole had been backfilled with boulders and cobbles and the water table returned to normal.

7. GROUNDWATER PRESSURE MONITORING & INTERPRETATION

Installation details

What will be discussed below is the interpretation of data recorded by two Adara electronic piezometers which were implanted in the natural ground at the location shown in Figure 1. Piezometer #1 (P1) was set at El 382.3m and Piezometer #2 (P2) at El 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 were monitored for 10 weeks before construction work began in order to establish baseline data and to gain confidence in their functioning. Then, after work started in October 2005, readings were taken on an intermittent basis throughout the following 12 months - a duration which covered the construction period.

Before getting into their responses to construction activity it is worth looking at how they behaved to an extraneous off-site repetitive event. Figure 8 shows the water pressure history recorded by P1 at 6 minute intervals over a period of seven days. It shows a weak pressure pulse of relatively constant intensity (20mm of head) entering the groundwater every morning at 7am and leaving at 5pm that same afternoon.

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. What we learn from this instantaneous and undamped transmission through the pore water is that we are dealing with a 2-phase system, just solids and water. If the medium had been 3-phase (solid, liquid and vapour) the entrained air would interfere 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.

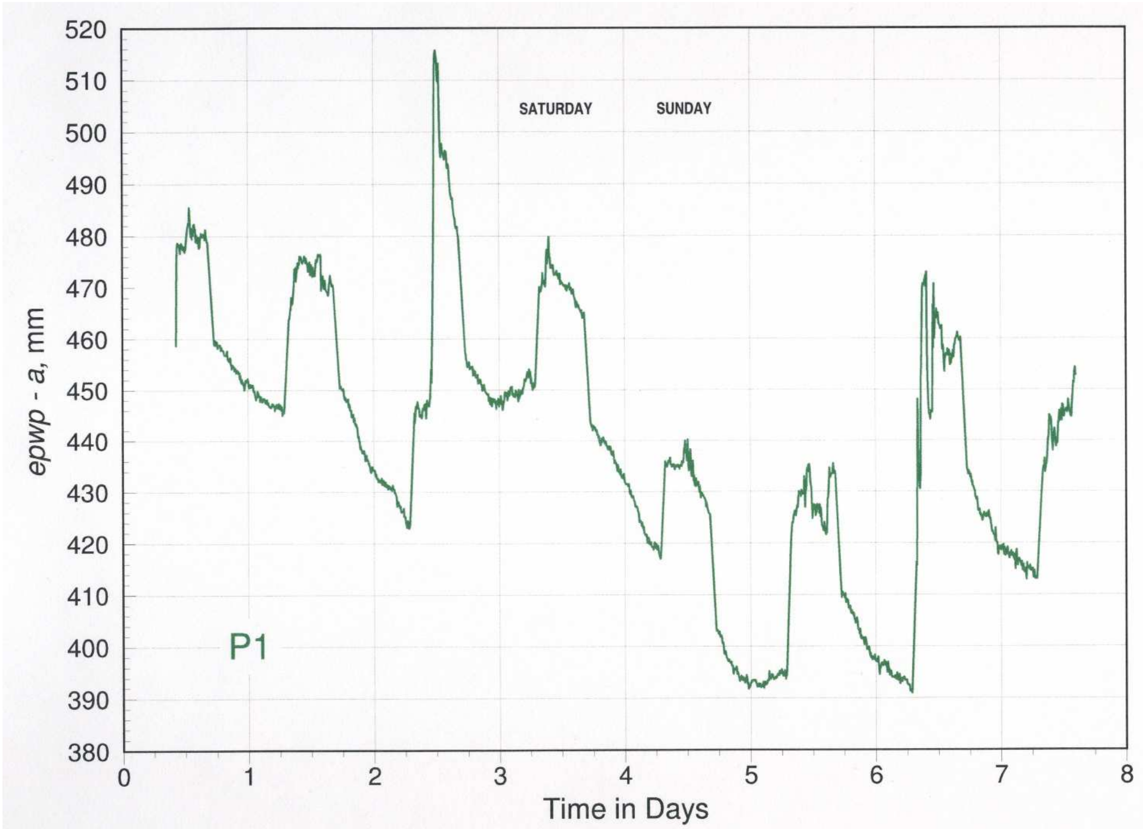

Figure 8 Extraneous ground water pressure pulses

The fact that we now know that the pore water in these clays exists as a physically unbroken fluid continuity (no air bubbles) makes it possible to understand how/why it is that whatever affects the water-phase of the soil in one place simultaneously affects the water pressures elsewhere.

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 I find of great 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. And, this observation alone confirms the axiom that when/where excess pore water pressure, that is, pressure in excess of hydrostatic, exists within a saturated soil seepage flow must also be taking place.

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

Paradigm under which the interpretation will proceed:

What follows is my 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.

In my work involving the h-method I take the following as being axiomatic:

1. 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.

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

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

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

5. Instrumentation cannot warn of future ground instability, but will signal when soil-structure movement is taking place.

These rules/axioms were first introduced in Ref 2. For the three-phase system, that is, partially saturated soils, see Ref 1.

Stabilization after installation trauma

On July 12th 2015 the piezometers were placed at predefined depths within the foundation clays. This was done using the Adara technique of encasing the delicate instrument inside a steel cone, and then using the CPT rig to push the cone-cum-piezometer assembly into the ground to the required depth. Data recorded immediately after installation of P1 showed water pressures rising to 115kPa (11.7m) within the next 50 minutes and then beginning a gradual decline. These high readings are a result of the cavity expansion shear strain-induced pore pressures and reflect the trauma experienced by the soil-structure as it is subjected to large straining imposed by the intrusion of the cone.

Artesian condition

Readings from P1 and P2 over the next 10 weeks prior to the commencement of construction showed that the groundwater was under an artesian affect. As depicted on the left side of Figure 5, the piezometric heads showed that an upward hydraulic gradient of 7.1% existed between them. This is not surprising given the nearby mountains and the layered nature of the foundation soils. This had only a minor effect on construction, but it did have a significant effect on foundation behaviour and instances of these will be discussed later.

A matter of epwp or epwp-a

In my earlier writing (Ref 2) I used "*epwp*" as an abbreviation for "excess pore water pressure" and there define this quantity as the "pressure in excess of hydrostatic caused by relative motion between the phases". By this definition when *epwp* = zero there is no surplus energy in the water phase and therefore neither hydraulic gradient nor seepage flow. But in the case of this particular site, where both piezometers are in an artesian flux, and consequently there is a background bias towards upward flow, I will introduce a parallel term "*epwp-a*" for the sake of consistency within my work. Whereas *epwp* is based on hydrostatic pressure, *epwp-a* is based on the vertical distance to the phreatic surface times the unit weight of water, γ_W. My reason for doing this is that what I am interested in here is isolating the pore pressure changes brought about by, and limited to, construction activity – and this new measure accomplishes that. So by applying this method the data from both piezometers can be brought into direct comparison on a common axis where the zero reading refers to the pre-construction *status quo ante* condition.

Piezometric record

The recorded data sets produced by the two piezometers from July 2005 to August 2006, a time which covered the period from the stabilization of the instruments to the end of construction, are shown plotted in Figure 9. These plots start on July 14th, three days after installation, and show that over the next six or seven weeks the *epwp-a* due to cavity expansion dissipate and both readings tend towards zero. Then, on October 1st the excavation of the weak and compressible soils above the desiccated crust gets underway and the pit is dewatered.

First let's confine our attention to P1. The excavation dewatering drawdown results in a drop of 22.9kPa being recorded on November 3rd, see point A (P_TA) in Table 1. It should be noted that there are insufficient readings during this period to be sure this is the minimum value, nevertheless it relates very well to a loss of hydrostatic head, that is, a fall in the water table level of 2.35m which is equivalent to 23.0kPa of pressure. Then after backfilling the hole and letting the water table reinstate itself, the December 1st pore pressure (P_TE) returned to just above the starting mark. I can't say, on the basis of the record, whether or not this additional 1.5kPa *epwp-a* is due to backfilling being placed which was in excess of the original ground load or to some other cause.

Now, scanning the P1 response during December 2005 through to the end of February 2006 we see a trace which is jagged, with sharp highs and lows – and must ask what does this mean in terms of soil behaviour. The instinctive answer is that these highs must be related to, or caused by, the ever increasing material loads hauled onto the site to make the building grow. But before we consent to the rationality of that surmise we are obliged to explain how weights added at ground level can have any effect on the groundwater? The fact is that the ground above the water table is a 3-phase system (solid, liquid, and vapour), and that the third (vapour/air) phase interrupts, and all but denies, the transmission of pressure through the water phase (see Ref 1). Without a physical justification we would therefore be obliged to conclude that pressures added to unsaturated ground could not affect the pore water pressure. But our experience would rebel against that notion.

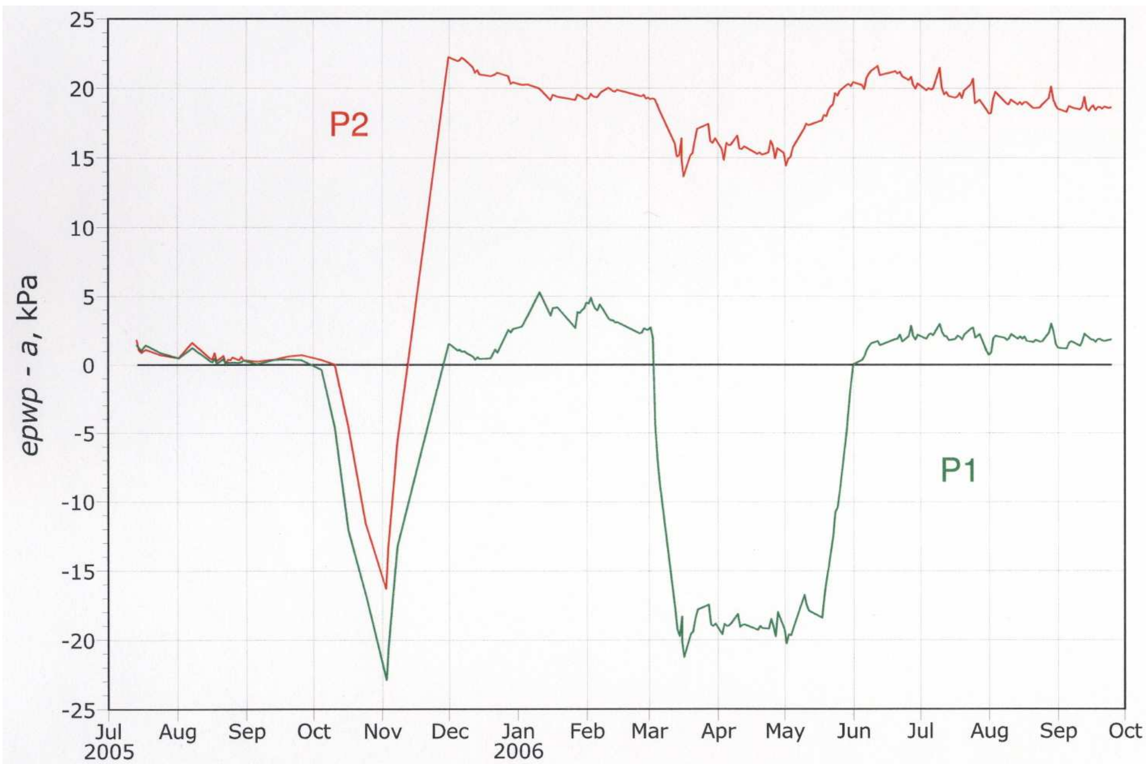

Figure 9 History of piezometer responses to construction

The reconciliation of practical experience and theoretical notion is as follows: New loading added to the dry ground above the water table is carried by the soil-structure down the soil column to a level which can sustain it. In so doing, this soil column is put into compression. The commensurate strain involves, of necessity, relative motion between the solid and water phases. As shown in Ref 2 Parts 1 and 2, such interaction between the phases causes water pressure to be generated ahead of the solid particle. And this is the rational explanation as to how the increase in loading on dry ground affects pore water in the saturated soil at depth.

Table 1
Excess Pore Water Pressure – above artesian level
measured on selected dates

Date	Point	P1	Point	P2	Unit	P2 - P1
November 3, 2005	**A**	-22.89	**B**	-16.32	kPa	6.58
November 8, 2005	**C**	-13.23	**D**	-5.79	kPa	7.44
December 1, 2005	**E**	1.49	**F**	22.23	kPa	20.73
March 3, 2006	**G**	1.89	**H**	19.25	kPa	17.35
March 17, 2006	**I**	-21.25	**J**	13.64	kPa	34.89

P1 signifies the presence of a hydraulic gradient within the system and consequently we know that water is flowing through the soil-structure. So what is happening is that water is seeping out of the soil, and this of course, tends towards the degradation of the potential. Depending on whether fresh load is added more quickly than seepage can reduce *epwp*, or the reverse is true will determine whether pore pressure subsequently rises or falls. For instance, during early December the rate at which surface loading was added was insufficient to prevent *epwp* dissipating almost completely by mid-month.

<u>P2 reacts differently from P1</u>

Let's temporarily leave P1 here at the end of February 2006 and switch our attention to its partner, P2, 12.9m directly below. A quick glance at the two traces on Figure 9 is enough to show that, immediately after construction activities start up in October, the two instruments respond very differently to the building contractor's activities. The reason for this disparity is simply the very different soil types in which these instruments are embedded. As shown in Figure 5, P1 resides in the near-lithified desiccated crustal material, just above the suction stiffened clay, whereas P2 is embedded in the normally consolidated plastic clay.

This means that if freshly imposed surficial loading does not exceed about 300kPa (see Figure 6) there is no reason for this geological unit to consolidate further beyond its existing state. Therefore, whatever relative motion is called for is strictly related to, and limited to, the elastic compression around P1. However, in the neighbourhood of P2 that same increase in loading will exceed the capacity of that viscoplastic soil to carry the additional load in its existing state of compaction, and in order to do so, must expel pore water so as to adopt a stronger soil-structure. That is, it must undergo consolidation – a structural adaptation which takes time.

Consolidation theory

Since it is simply the soil consolidation mechanism that plays such a significant role in the behaviour of P2 I think I should make the following points. Herein, I deal with consolidation in fundamental physical terms, that of solid particles finding a tighter packing arrangement amongst their neighbours, and of the time and energy this takes while the displaced pore water finds its way out of the system. This approach appeals to me far more than employing the thermodynamic equation we have adopted on the basis of the tight math-physics analogy between heat flow and seepage flow through a conductive medium. My problem with our wholehearted adoption of this convenience is that when a copper plate is subjected to an electrical or thermal potential gradient that conductor will be the same elemental continuum before and after the test; a soil specimen will not. The soil will be changed by the procedure. And this is a case in point where assuming that soil specimens represent elemental parts of a continuum is obviously wrongminded.

The mechanistic approach to consolidation

Looking at the consolidation of soil in mechanical/physical terms is more enlightening for a geotechnical engineer, rather than our current practice of perceiving it through the lens of a truly elegant, if borrowed, mathematical analogy. The need for soil to consolidate is the consequence of it being asked to carry a load increment which exceeds the competence of its current soil-structure. The soil responds to the demand for force equilibrium ($\Sigma V=0$) by expelling water from its voids in order that the solid-phase may contract its interparticle contacts into a formation which can support the additional load. This takes time because of the hydrodynamic interplay involved.

What actually goes on is that in a futile effort to prevent the invasion of its space, the pore water exerts hydrodynamic drag forces (D_F) against the intruding particles. The source of energy required to empower the drag resistance comes from the energy (potential) lost by the downward movement of the solids. To allow this phase replacement/exchange to develop/persist it is necessary that, concurrently, any water expelled from the voids be able to exit the system. As explained in Ref 3, the summation of D_F is most conveniently expressed as a Seepage Force (S_F), acting in the same direction. S_F is in effect a hydraulic gradient acting on a volume of water, and it is this S_F which enables the latent water to exit the system. In this way consolidation may be seen as a rather elegant *quid pro quo* between natural needs.

Perhaps it will help to show how, in this context, the ledger is balanced with regard to Work Done and Energy Expended: Within this 2-phase system the work done by the downward movement of the solid phase (weight times distance, F * L) is transposed into energy generation (pressure times volume, $F/L^2 * L^3$) in the liquid phase over the depth where relative motion takes place. Therefore, the source of energy to support/sustain the exit gradient comes from the potential shed by the descending solid particles.

In the course of time, as the expulsion of pore water permits the soil-structure to form into the necessarily more compact particle arrangement, the solid-phase will become capable of carrying more of the new surface loading, so that the magnitude of the S_F thereafter required to achieve vertical equilibrium is diminished. Eventually, further relative motion between the phases will no longer be necessary under this load increment. Then the S_F, with its attendant hydraulic gradient and *epwp* will depart the scene, thus giving way to the hydrostatic (here, phreatic) condition again.

But, a cautionary note: This tighter packing of the solids has the unavoidable consequence of reducing the conductivity of the soil, significantly altering the transmissivity, thereby affecting the soil's subsequent behaviour. This is one of the reasons I object to the notion of a soil, a multiphase medium, being (mis)represented as if it were a continuum.

<u>P2 compared with P1</u>

Right from the time the contractor moved equipment onto the site the two piezometer traces went their separate ways despite being subjected to the same sequence of events. Referring to the data shown on Table 1 it may be seen that about a month after the start of drawdown Point B (P_TB), the November 3, 2005 pressure at P2 fell less than the higher piezometer (P_TA) by 6.6kPa, and then regained another 10.5kPa (P_TD) of relative pressure in 5 days. At that rate it would be reasonable to expect pore water condition at P2 to be back to zero within about another 4 or 5 days, that is, before mid-November. What happened next caught me by surprise. How could it be that P_TF on December 1st could be standing at 22.2kPa while at the same time the instrument vertically above it is reading close to its zero value? It could only be that the drawdown caused a fundamental change in the foundation's constitution.

The explanation is as follows: The excavation and dewatering of the pit resulted in significant rebound of the normally consolidated clay in which P2 resides. The water required to allow those voids to expand/dilate was readily available by lateral flow from outside the building footprint under the inward hydraulic gradient which was created by the localized drawdown. Now, with the void water quickly equilibrated with the unloaded condition, the subsequent replacement of the load produced a full *epwp* response - as if these clays were being subjected to fresh loading for the first time. Incidentally, I'm inclined to think that this rebound might have been confined to the zone beneath the crust which I referred to earlier as having been stiffened by capillary action.

But then another question arises: Why, when it took only a matter of days for the water to enter the strata did it take, as we will see, a matter of years for this same water to be expelled again? The reason is simple. The hydraulic gradient under which water entered the foundation area in November was lost entirely when the hole was allowed to refill with water; thereafter this added void water could only exit the system by seepage flow energized by work done on the system. This flow rate and the energy source will be clarified subsequently herein.

The pressures changes in the ground resulting from the excavation and refilling of the pit, and the second drawdown, prove useful in understanding the concurrent pore water pressure changes. These are listed in Table 2 for the various phase states which are pertinent. The unit weights were calculated for void ratio e = 0.32, although at the design stage I guesstimated that the void ratio of the backfill layer would be 0.35. The reason for this reassessment will be explained later. The column height of overburden materials was taken as 2.35m and this comes from the minimum drawdown necessary to expose the "reliable top of the crust" as shown on Figure 5 (2.05m) plus an additional 0.3m for the drain intake level being that much lower.

Table 2
Compressive Stress
at base of 2.35m column of material

Material	**Unit weight, kN/m^3**		**Base Pressure**
Water	γ_W	9.81	23.0 kPa
Dry backfill	γ_D	19.7	46.2 kPa
Saturated backfill	γ_S	22.1	51.8 kPa
Buoyant backfill	γ_B	12.3	28.8 kPa

It should be noted that because the foundation strata are layered and of widely varying stiffness I did not attempt to calculate the theoretical diminishment of normal pressures with depth. Consequently, both piezometers are treated as if they were subjected to the same normal stress changes at any stage.

Now looking at the values of *epwp-a* listed in Table 1 we see that the value for P2 on December 1st (P_TF) is 22.2kPa. This is sensibly close to the pressure equivalent to reinstatement of the natural water table (23.0kPa), the difference (0.8kPa) being attributable to some amount of seepage occurring before then.

As stated earlier the P1 trace in the three months after allowing the water table to recover (December 2005, January and February 2006) is a jagged series of peaks and valleys. This shows the *epwp-a* caused directly by loading added to the ground surface, and their subsequent dissipation by seepage flow. In contrast the P2 trace is a more gradual loss of the high rebound potential as re-consolidation proceeds. Here the effects of the episodic additions of surface loading are subsumed beneath the ongoing consolidation *epwp*.

At the beginning of March the local water table was drawn down for a second time, on this occasion to enable installation of underground facilities "in the dry". The drawdown was the same as before (2.35m), but this time we have more data. Table 1 shows that P1 recorded a drop of 23.1kPa from P_TG to P_TI, a value which agrees with the pressure equivalent of the reduced hydrostatic column, 23.0kPa. It is important to note here that P1 is responding solely to a water-phase change, and this, without showing any evidence of relative motion between the phases. Had some movement of the soil-structure of the crust been required to respond to this event, that would have shown up as some amount of *epwp* which would have resulted in a smaller drop in the P1 reading.

The trace for P2 over the same period is quite different and informative on two counts:

a. The pressure drop from P_TH to P_TJ is 5.6kPa which represents the difference between a column of saturated backfill and one of dry backfill. And that is just as it should be because while the water table was in its natural position the weight of that fill was γ_S times its height, but after drawdown, when the fill was no longer submerged its weight became γ_D times its height. The fact that the numbers work out so well is really not that impressive; this it is because I used these two readings to back-calculate the void ratio of the boulder layer, causing me to change my design estimate of 0.35 to 0.32 in the process.

b. The amount of rebound was, for about a month, able to interrupt ongoing consolidation in the vicinity of P2. This temporary quieting/suppression of that background *epwp* allowed the subsumed effects of the changing surface loading to be uncovered. This outcome can be seen in the clear similarity of the traces for the two instruments during this period, where until mid-April the peaks and valleys are about the same height. But from then on they become more subdued as background consolidation starts to overtakes rebound.

The quite unusual circumstances of this drawdown, together with the simultaneous measurements of *epwp* in two soils of distinctly different stiffness to increasing surface loading turned out to provide an unique opportunity for checking the basic axioms on which my hydrodynamic approach (h-method or h-theory) rests.

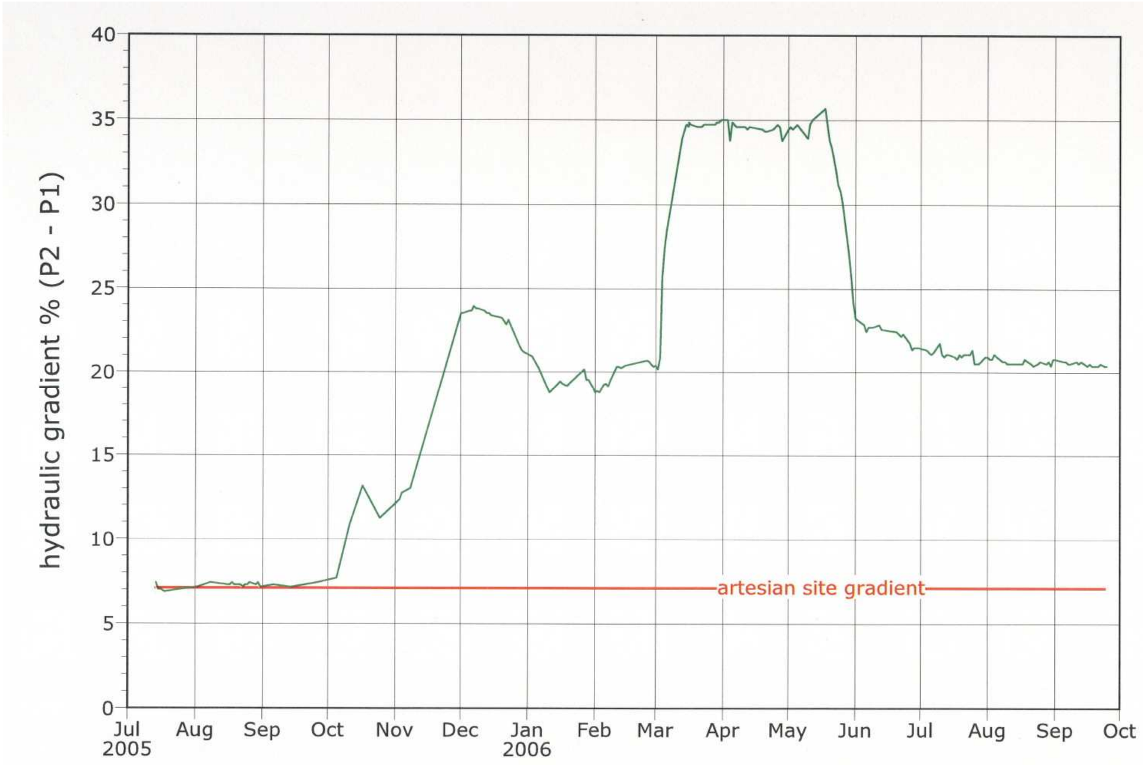

Figure 10 Hydraulic gradient P2 – P1

Essentially what the relationship between the two piezometers is telling us is the magnitude of the hydraulic gradient existing over the depth of soil between them. So the chart shown on Figure 10 was made to clarify this value over the construction period.

As may be seen the prevailing artesian condition forms the baseline, and unsurprisingly, the flow is always upwards. It is during the second drawdown that the hydraulic gradient is greatest, and holds relatively constant, at ~35%.

Of interest is how, as additional construction loading tails off, the gradient takes on the rough appearance of simple consolidation. And here it becomes obvious that at this rate of decline it will take a matter of years for *epwp* to fully dissipate by seepage flow.

<u>Post-construction readings</u>

Over the years following construction, whenever I passed through Vernon, I made a point of dropping in at the building to check the foundation slab for any signs of concrete cracking. On those occasions, if I had a readout device with me, I would check the piezometers too. Those readings are shown on Figure 11. Here the time scale shows dates, while the piezometric heads are plotted directly against elevation. This latter scale differs from the one I used in Figure 9 and was chosen in order to separate the data points for clarity. Also shown on Figure 11 is the water table and the zero (artesian) elevation for the two piezometers.

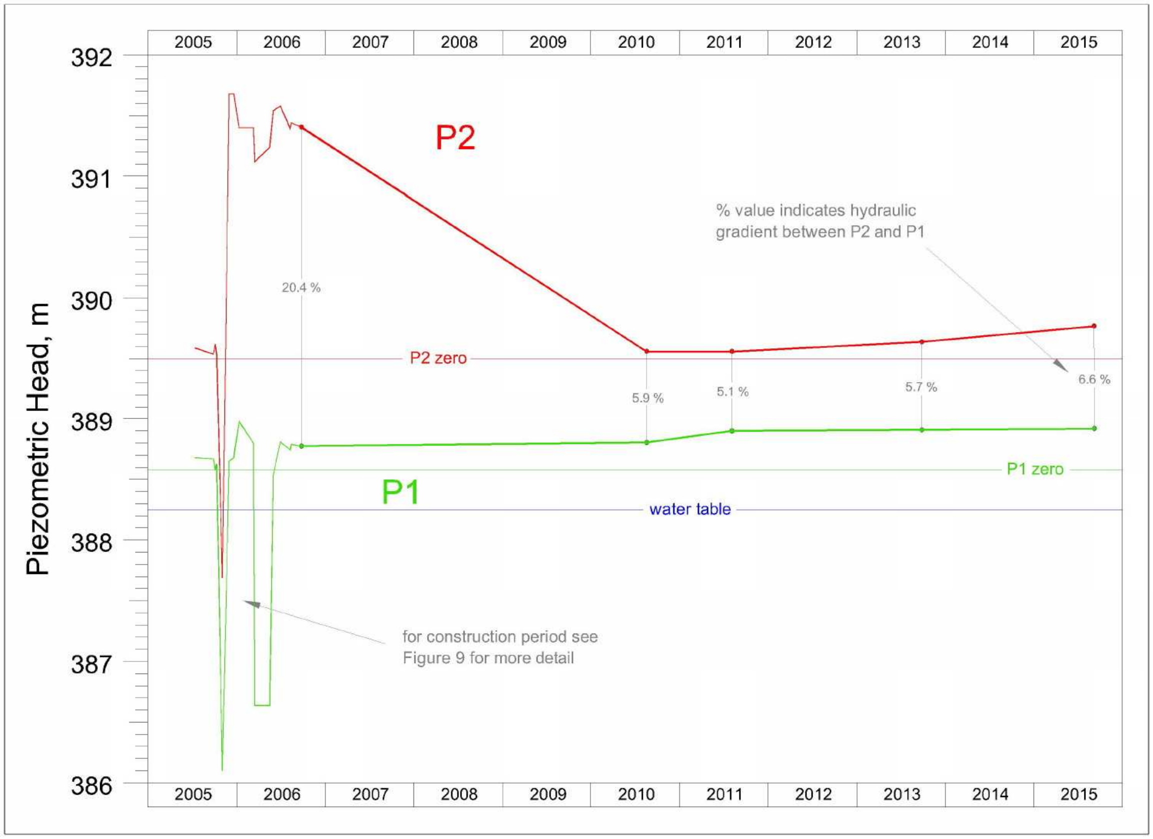

Figure 11 History of piezometer responses from 2005 to 2015

I was struck by the fact that both instruments were registering an increase in pore pressure. When I visited the site to read the 2015 data I showed the 2013 plot to the Owner (Marty Steele) and asked him if anything had been added to the site loading which could account for the water pressures being now above their pre-construction levels. His immediate response was "Could it have to do with the construction of the Casino?" This rather large structure was built on the neighbouring property to the north sometime after completion of work at this/our site. I cannot think of a better explanation and therefore I have adopted his.

So, what these few readings suggest to me is that, just like the off-site pulses discussed earlier (see Figure 8), these were transmitted through the water-phase, and had their origin in relative motion in the foundation strata beneath the Casino. It is likely significant that that structure was built following preloading of its foundations, and without removal of the upper weak and compressible strata, contrary to the procedure used at our site.

8. STRUCTURAL PREFORMANCE

Little needs to be said in this regard other than to draw attention to the photographic evidence offered in Figure 12. This shows the only crack in the entire 7,000 square metres of concrete flooring after 10 years of commercial operation.

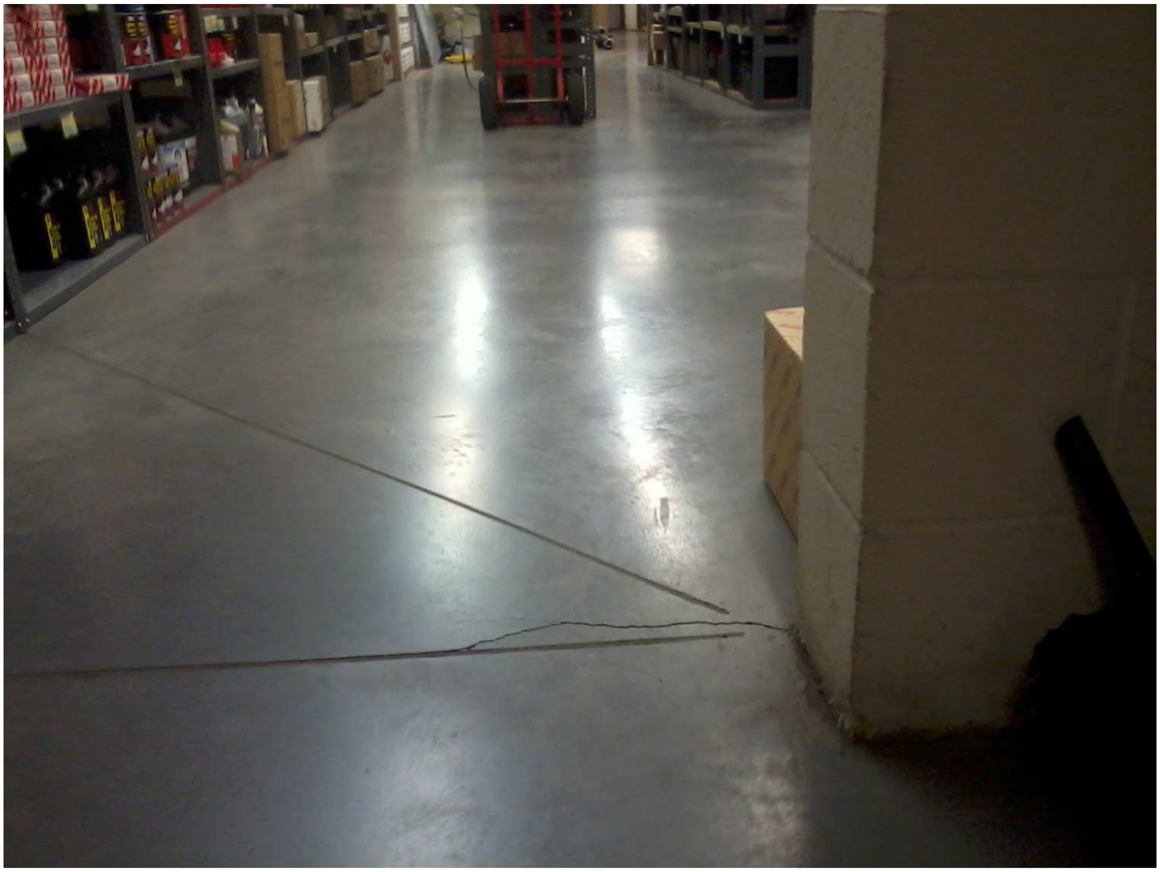

Figure 12 Only floor crack in concrete slab

9. CONCLUSIONS

From a practical 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.

To my mind the measured behaviour of the complex two-phase soil-structure at this site is proof positive that the hydrodynamic approach (h-method) is valid, and furthermore, demonstrates the diagnostic and explanatory power of this theory. For instance: Is there another way to explain how the surface loads added above the water table can show up on the piezometers? Or, could the piezometric data recorded during, and after, the two separate drawdown events be accounted for more simply? I think not.

10. ACKNOWLEDGEMENTS

I wish to thank David Carrier and Professor James Mitchell for their detailed technical reviews, and guidance towards better presentation. Both reviewers agree that the fundamental principle/axiom on which h-theory rests is correct.

11. REFERENCES

1. Compacted Earthfill is a 3-phase Material (2016), William E Hodge
http://www.phoenix-hodge.com/News%20Items/Final%20Text%20January%202016.pdf

2. Liquefiable Materials and their Treatment by Vibro-Draining (1998), BiTech, W. E. Hodge
http://www.phoenix-hodge.com/monograph.html

3. the Water in the Soil, Parts 1 and 2 (2010, 2011), Geotechnical News, Bill Hodge
http://www.phoenix-hodge.com/GN%20SIX.html

CONE INTERPRETATION OUTPUT   Code written by W.E. Hodge, P.Eng. as an interpretation aid for his own use only.
Based on U.B.C. research publications, mainly Robertson & Campanella, 1988.

07:11:05 09:17 CONETEC
Toyota site   20 Ton St 150   CPT05-01

Water table Depth (m)	1.25				
Stratum Number	1	2	3	4	5
Depth to Stratum Base	.9	3.0	6.5	19.7	50.0
Moisture Content	.15	.50	.50	.70	.65
Cone Factor	15.	15.	11.	12.	15.

TOP OF SOIL m	ft	THICKNESS m	ft	SOILTYPE UBC	AVE Q MPa	Bq	COHESION kPa	psf	SVE kPa	PHI deg	DR %	N	TOP EL m
.00	.0	1.50	4.9	organic material	.0	.010			14.			0	389.50
1.50	4.9	.55	1.8	sensitive fine grained	.4	.030	23.	470.	25.			2	388.00
2.05	6.7	.05	.2	silty clay to clay	.5	.017	29.	607.	27.			3	387.45
2.10	6.9	.05	.2	clayey silt to silty clay	1.0	.013	62.	1304.	28.			5	387.40
2.15	7.1	.05	.2	silty sand to sandy silt	2.1	-.005			28.			7	387.35
2.20	7.2	.10	.3	sandy silt to clayey silt	2.0	-.007			29.			8	387.30
2.30	7.5	.10	.3	silty sand to sandy silt	2.5	-.010			29.			8	387.20
2.40	7.9	.05	.2	sandy silt to clayey silt	1.6	-.021			30.			6	387.10
2.45	8.0	.10	.3	clayey silt to silty clay	1.0	-.040	61.	1269.	30.			5	387.05
2.55	8.4	.20	.7	sensitive fine grained	.7	-.012	42.	879.	31.			3	386.95
2.75	9.0	.40	1.3	clayey silt to silty clay	1.0	-.016	76.	1591.	33.			5	386.75
3.15	10.3	.20	.7	silty clay to clay	1.0	-.025	94.	1954.	36.			7	386.35
3.35	11.0	.15	.5	clayey silt to silty clay	1.2	-.020	111.	2315.	37.			6	386.15
3.50	11.5	.20	.7	silty clay to clay	1.5	-.013	135.	2820.	38.			10	386.00
3.70	12.1	.25	.8	clayey silt to silty clay	1.4	-.006	129.	2687.	40.			7	385.80
3.95	13.0	.20	.7	silty clay to clay	1.3	.012	116.	2418.	41.			9	385.55
4.15	13.6	.35	1.1	clayey silt to silty clay	1.3	.041	114.	2387.	43.			6	385.35
4.50	14.8	.10	.3	clay	1.2	.073	109.	2266.	45.			12	385.00
4.60	15.1	.65	2.1	silty clay to clay	1.0	.213	95.	1976.	47.			7	384.90
5.25	17.2	.15	.5	clayey silt to silty clay	1.0	.341	96.	2015.	50.			5	384.25
5.40	17.7	.10	.3	silty clay to clay	1.0	.353	94.	1971.	51.			7	384.10
5.50	18.0	.05	.2	clay	1.0	.370	90.	1886.	51.			10	384.00
5.55	18.2	.15	.5	silty clay to clay	1.1	.303	98.	2057.	52.			7	383.95
5.70	18.7	.80	2.6	clayey silt to silty clay	1.1	.396	99.	2058.	55.			5	383.80
6.50	21.3	.10	.3	silty clay to clay	.8	.557	70.	1459.	58.			6	383.00
6.60	21.7	.65	2.1	clayey silt to silty clay	.8	.506	61.	1276.	60.			4	382.90
7.25	23.8	.10	.3	sensitive fine grained	.7	.621	55.	1148.	63.			3	382.25
7.35	24.1	.05	.2	clayey silt to silty clay	.7	.571	56.	1167.	63.			4	382.15
7.40	24.3	.10	.3	sensitive fine grained	.7	.571	55.	1141.	63.			3	382.10
7.50	24.6	.25	.8	silty clay to clay	.7	.541	52.	1089.	64.			4	382.00
7.75	25.4	.15	.5	sensitive fine grained	.6	.614	47.	991.	66.			3	381.75
7.90	25.9	.05	.2	silty clay to clay	.6	.613	46.	963.	66.			4	381.60
7.95	26.1	.50	1.6	sensitive fine grained	.6	.669	43.	889.	68.			3	381.55
8.45	27.7	.15	.5	silty clay to clay	.6	.595	40.	838.	70.			4	381.05
8.60	28.2	.90	3.0	sensitive fine grained	.6	.620	42.	877.	72.			3	380.90
9.50	31.2	.10	.3	silty clay to clay	.6	.646	42.	875.	75.			4	380.00
9.60	31.5	.05	.2	clayey silt to silty clay	.7	.380	48.	999.	76.			3	379.90
9.65	31.7	4.85	15.9	sensitive fine grained	.6	.692	43.	895.	90.			3	379.85
14.50	47.6	.20	.7	clayey silt to silty clay	.7	.577	50.	1053.	104.			4	375.00
14.70	48.2	.25	.8	sensitive fine grained	.7	.681	50.	1054.	105.			4	374.80
14.95	49.1	.10	.3	clayey silt to silty clay	.8	.661	55.	1141.	106.			4	374.55
15.05	49.4	.10	.3	sensitive fine grained	.8	.688	54.	1131.	107.			4	374.45
15.15	49.7	.35	1.1	clayey silt to silty clay	.8	.624	57.	1197.	108.			4	374.35
15.50	50.9	.30	1.0	sensitive fine grained	.8	.665	56.	1162.	110.			4	374.00
15.80	51.8	.20	.7	clayey silt to silty clay	.8	.605	57.	1191.	111.			4	373.70

TOP OF SOIL		THICKNESS		SOILTYPE	AVE Q	Bq	COHESION		SVE	PHI	DR	N	TOP EL
m	ft	m	ft	UBC	MPa		kPa	psf	kPa	deg	%		m
16.00	52.5	.05	.2	sensitive fine grained	.8	.709	52.	1096.	112.			4	373.50
16.05	52.7	.20	.7	clayey silt to silty clay	.8	.638	57.	1190.	113.			4	373.45
16.25	53.3	.05	.2	sensitive fine grained	.8	.602	54.	1132.	113.			4	373.25
16.30	53.5	.10	.3	clayey silt to silty clay	.8	.648	56.	1173.	114.			4	373.20
16.40	53.8	.05	.2	sensitive fine grained	.8	.637	54.	1134.	114.			4	373.10
16.45	54.0	.05	.2	clayey silt to silty clay	.8	.659	54.	1125.	114.			4	373.05
16.50	54.1	.70	2.3	sensitive fine grained	.8	.685	52.	1084.	117.			4	373.00
17.20	56.4	.35	1.1	clayey silt to silty clay	.8	.657	53.	1109.	120.			4	372.30
17.55	57.6	.20	.7	sensitive fine grained	.8	.701	52.	1077.	121.			4	371.95
17.75	58.2	.40	1.3	clayey silt to silty clay	.8	.662	56.	1162.	123.			4	371.75
18.15	59.6	1.00	3.3	sensitive fine grained	.8	.708	49.	1029.	127.			4	371.35
19.15	62.8	.10	.3	clayey silt to silty clay	.8	.716	54.	1125.	130.			4	370.35
19.25	63.2	.15	.5	sensitive fine grained	.8	.724	53.	1117.	131.			4	370.25
19.40	63.7	.05	.2	clayey silt to silty clay	.8	.738	52.	1080.	131.			4	370.10
19.45	63.8	.05	.2	sensitive fine grained	.8	.804	50.	1039.	131.			4	370.05
19.50	64.0	.20	.7	clayey silt to silty clay	.8	.650	54.	1120.	132.			4	370.00
19.70	64.6	.05	.2	sandy silt to clayey silt	.9	.644			133.			4	369.80
19.75	64.8	.20	.7	sensitive fine grained	.9	.723	44.	912.	134.			4	369.75
19.95	65.5	.05	.2	sandy silt to clayey silt	.9	.657			134.			4	369.55
20.00	65.6	.20	.7	clayey silt to silty clay	.9	.714	44.	921.	135.			4	369.50
20.20	66.3	.20	.7	sandy silt to clayey silt	.9	.731			136.			4	369.30
20.40	66.9	.05	.2	clayey silt to silty clay	.9	.739	44.	911.	137.			4	369.10
20.45	67.1	.15	.5	sandy silt to clayey silt	.9	.693			138.			4	369.05
20.60	67.6	.05	.2	clayey silt to silty clay	.9	.685	44.	917.	138.			4	368.90
20.65	67.8	.10	.3	sandy silt to clayey silt	.9	.660			139.			4	368.85
20.75	68.1	.15	.5	clayey silt to silty clay	.9	.710	43.	901.	139.			4	368.75
20.90	68.6	.10	.3	sandy silt to clayey silt	.9	.712			140.			4	368.60
21.00	68.9	.05	.2	clayey silt to silty clay	.9	.731	44.	925.	141.			4	368.50
21.05	69.1	.85	2.8	sandy silt to clayey silt	.9	.727			143.			4	368.45
21.90	71.9	.05	.2	sensitive fine grained	.9	.818	44.	927.	146.			5	367.60
21.95	72.0	7.55	24.8	sandy silt to clayey silt	1.2	.722			168.			5	367.55
29.50	96.8	.10	.3	clayey silt to silty clay	1.5	.657	77.	1603.	191.			7	360.00
29.60	97.1	5.00	16.4	sandy silt to clayey silt	1.5	.735			206.			6	359.90
34.60	113.5	.05	.2	silty sand to sandy silt	1.6	.765			221.			5	354.90
34.65	113.7	.05	.2	sandy silt to clayey silt	1.6	.657			222.			7	354.85
34.70	113.9	.05	.2	clay	.8	1.284	25.	516.	222.			8	354.80
34.75	114.0	9.75	32.0	sandy silt to clayey silt	1.8	.701			251.			7	354.75
44.50	146.0	.05	.2	silty sand to sandy silt	2.1	.664			280.			7	345.00
44.55	146.2	.05	.2	sandy silt to clayey silt	2.1	.664			280.			8	344.95
44.60	146.3	.05	.2	clay	1.0	1.400	28.	593.	281.			10	344.90
44.65	146.5	1.35	4.4	sandy silt to clayey silt	2.0	.679			285.			8	344.85
46.00	150.9	.05	.2	silty sand to sandy silt	2.3	.551			289.			8	343.50
46.05	151.1	.45	1.5	sandy silt to clayey silt	2.2	.670			290.			9	343.45
46.50	152.6	.10	.3	silty sand to sandy silt	2.1	.674			292.			7	343.00
46.60	152.9	1.05	3.4	sandy silt to clayey silt	2.1	.696			295.			8	342.90