Using a Leaky Swimming Pool for a Huge Falling-Head Permeability Test

Robert P. Chapuis
Ecole Polytechnique, Montreal, QC, Canada,

ABSTRACT
Despite recent repairs, a heated swimming pool had major leaks. To determine the possible position of the leak or leaks, a full scale falling-head permeability test was performed. All the valves for circulating hot and cold water were closed to monitor the variation with time of the water level in the pool. The velocity graph, which represents the conservation equation in variable-head permeability tests, was used to determine the position of leaks. Subsequent excavations found broken pipes and connections, their failure being related to high differential settlement caused by poor compaction of pipe foundations.

RÉSUMÉ
Malgré des réparations récentes, une piscine chauffée avait des fuites majeures. Pour déterminer la position possible de la fuite ou des fuites, un essai de perméabilité à niveau descendant à grande échelle a été réalisé. Toutes les valves pour la circulation de l'eau chaude et de l'eau froide ont été fermées afin de suivre la variation dans le temps du niveau d'eau dans la piscine. Le graphique des vitesses, qui représente l'équation de conservation dans les essais à charge variable, a servi à déterminer la position des fuites. Des excavations subséquentes ont trouvé des tuyaux et des raccords brisés, leur rupture étant reliée à d'importants tassements différentiels causés par le mauvais compactage des fondations des tuyaux.

1 INTRODUCTION

In the Montreal area, outside non-heated swimming pools are operated for two summer months most of the time. Using preheated water may extend significantly the operation period. This paper presents the case history of a 40-year old swimming pool. Major repairs and improvements were done about 20 years ago, including repairs of walls, pumping house, full replacement of water pipes and filtration system, and addition of heating system and pipes. After completion of this work, the first pool operation indicated injected volumes higher than returning volumes. The leakage rate was in the range of 350 to 700 m^3 per day. The pool was emptied. Several leaking pipes and defective connections were detected and repaired. Although it was thought that all the leaks had been repaired, the registered inflow and outflow volumes were still unbalanced, the leakage being estimated between 3 and 6 m^3/h. The position of the remaining leaks was unknown because they were along underground buried pipes at elevations between 42 and 46 m (Fig. 1).

The major questions can be summarized as follows. Was it possible to determine the location(s) of the leak(s)? Was it possible to re-open trenches of limited extent to check the condition of pipes and connections, and then repair them to stop the leak(s)? What had caused the leak(s)?

To determine the elevation of the leak(s), the swimming pool was used to perform a variable-head permeability test, measuring the leakage rate as a function of the water level. The paper describes the test, the interpretation of data, the observations of pipes and couplings in the trenches, and documents the reasons for having major leaks. After the repairs, a test was performed to verify whether all pipes were watertight, before infilling the pool: the test was successful.

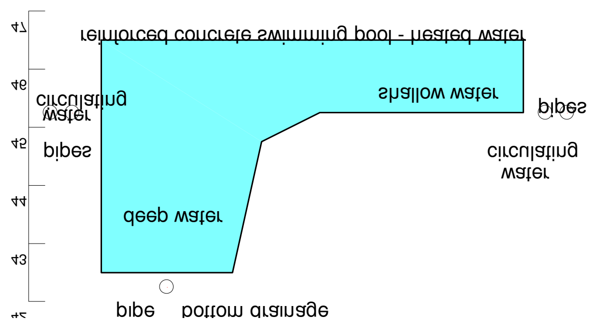

Figure 1. Simplified sketch of the swimming pool.

2 PERMEABILITY TEST

2.1 Performing the Test

A simplified cross-section of the swimming pool appears in Fig. 1. To understand the analogy with a field permeability test performed using either an open borehole and packers, or a monitoring well (MW), the pool represents the variable-head tank (external tank or volume of water in the MW pipe), the pipes represent the drill stem or the MW pipe, and water is seeping from the water tank into the ground after having passed through the connecting pipes. For running the variable-head permeability test, the swimming pool was filled up to a fixed level, then allowed to leak after all valves were closed. The testing sequence is described in Fig. 2. The leaks lowered the water level in the pool. The water level drop was monitored versus elapsed time *t* and used to calculate the leaking flow rate.

2.2 Interpreting the Test

The data of variable-head field permeability tests can be interpreted using different methods. Chapuis (1998) classified the methods into three groups that aim to provide: (1) the local hydraulic conductivity (*k*) of either an aquifer or an aquitard; (2) the local *k* value and delayed compressibility of an aquitard; (3) the transmissivity (*T*) and the storativity (*S*) of an aquifer. In group 1 methods, the influence of the solid matrix deformation is neglected in the mass conservation equation (e.g., Lefranc 1936, 1937; Hvorslev 1951; Bouwer and Rice 1976; Bureau of Reclamation 1977). Methods of group 2 consider an aquitard with a solid matrix deformation that is elastic but delayed, and influences the mass conservation equation through consolidation (e.g., Gibson 1966). Methods of group 3 assume that the aquifer has an immediate elastic solid matrix strain, which influences the mass conservation equation (e.g., Cooper et al. 1967). For the same test data, the various methods are known to yield *k* values that may differ by two orders of magnitude (Herzog and Morse 1990; Herzog 1994).

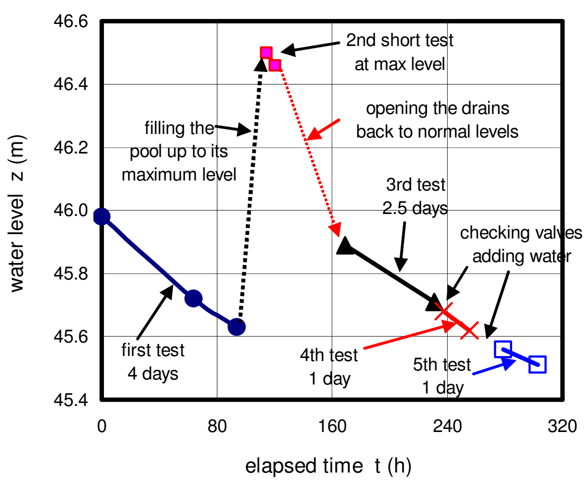

Figure 2. Testing periods and filling periods

Here, the pool leakage corresponds to water seeping into an aquifer material, namely the crushed stone around the leaking pipes, which explains the high flow rates. Consequently, the group 2 methods are irrelevant. And, the group 3 methods can be ruled out because the underlying theories have been shown to be wrong for variable-head (slug) tests (Chapuis 1998) and also pulse tests (Chapuis and Cazaux 2002). The basic solution for group 3 methods is that of Cooper et al. (1967) for slug tests. This solution was derived by analogy with a heat conduction problem. However, the slug test involves mass and energy transfers whereas the conduction test involves energy transfer without mass transfer and therefore, the analogy is incorrect. Also, the thermal test mathematically resembles some pulse test and not a slug test. Besides, the definition of parameter α in the solution is wrong due to confusion when translating a heat capacity ratio as a hydraulic storativity ratio. Physically, using isotropic elasticity, the influence of solid matrix strain was shown to be so small that usually it does not affect slug test data (Chapuis 1998). Only for soft soils the influence of strain may be important (Chapuis 1999), the reason why the group 2 specific methods were developed. Numerical modeling of slug tests has confirmed the findings in math and physics (Chapuis 1998). The basic solution of group 3 for slug test (Cooper et al. 1967) may yield a *k* value that is wrong by a factor 5 (Chapuis and Chenaf 2002) or 10 (Poirier et al. 2004), and an *S* value that is wrong by a factor ranging between 200 and 50 000 (Chapuis and Chenaf 2002). Also, the basic solution of group 3 for pulse tests (Bredehoeft and Papadopulos 1980) was shown to be wrong, by two orders of magnitude for *k* values, and four to eight orders of magnitude for *S* values (Chapuis and Cazaux 2002). One of the consequences of those recent findings is that the standard D4104 (ASTM 2008a) for slug tests was not renewed in 2002 by ASTM, and the interpretation method of standard D4631 for pulse tests (ASTM 2008b) is likely to be modified.

Therefore, this paper considers only interpretation methods of group 1, including the velocity graph method which is retained in standards (e.g., AFNOR 1992; CAN/BNQ 1988, 2008) for slug tests.

2.3 Retained Interpretation Method

When the soil is at constant saturation and its solid matrix strain has a negligible influence in the water mass conservation equation, the local groundwater mass conservation equation yields the Laplace equation, $\nabla^2 h = 0$ where *h* (m) is the hydraulic head. Its solutions are called harmonic functions in mathematics. For any field permeability test, the Laplace equation is solved only for the flux value at the boundary where the hydraulic head is the measured parameter. For a field permeability test, this partial solution is

$$Q_{inj} = Q_{soil} = kcH \qquad [1]$$

In Eq. (1), Q_{inj} is the injected flow rate measured in the water tank (here the pool, usually the MW casing), Q_{soil} is the flow rate into the soil, *c* is the shape factor of the test, *k* is the hydraulic conductivity, *H* is the difference in hydraulic head existing between the tested soil or rock (at rest) and the injection tank. In a variable-head test, Q_{inj} is obtained as the product of the internal cross-section area, *A* of the injection tank by the water downward velocity, *dH*/*dt*, in the tank

$$Q_{inj} = -A\frac{dH}{dt} \qquad [2]$$

The minus sign results from the physics. A decrease of *H* with time ($dH/dt < 0$) means that water is injected from the water tank into the ground, whereas an increase ($dH/dt > 0$) means that water seeps from the ground into the tank. Then, the test differential equation is obtained by combining Eqs. (1) and (2)

$$\frac{dH}{dt} = -\frac{ck}{A}H \qquad [3]$$

Its integration yields the classical equation of Hvorslev (1951)

$$\ln\left(\frac{H_1\,(t=t_1)}{H_2\,(t=t_2)}\right) = -\frac{kc}{A}(t_1 - t_2) \qquad [4]$$

where H_1 and H_2 are the differences in hydraulic heads at times t_1 and t_2 respectively. As a result, a plot of ln (H) against time t provides directly the k value.

Several equations are available for the shape factor c, which depends on the geometry of the injection zone, and whether impervious or recharge boundaries are close to the tested aquifer volume (e.g., Chapuis 1989). In the case of a swimming pool viewed as a tank injecting water into the ground through pipe defects of unknown geometry, the c value is unknown. Knowing the c value would provide the k value for the tested "soil", which is the 0-20 mm crushed stone below, around and above the buried pipes. Note that the measured leak flow rate could be easily evacuated at atmospheric pressure by a trench filled with crushed stone. Here, our interest in the permeability test for the leaking pool is not to find the k value of the crushed stone, but to find the position (elevation) of the leak. Is this possible? The answer is hidden in the above equations. A short theoretical development is needed to extract this information.

In Eq. (4), H is the difference in total head between the water tank (e.g., water in the MW pipe or here the swimming pool), and the water around the injection zone or the leaks. Note that after passing the leaky crack, water rapidly reaches atmospheric pressure in the pervious crushed stone: therefore H may be viewed as the difference between the pool water elevation and the leak elevation. However, the position of the leak is unknown, and thus the real H value at any time t is unknown. Therefore, some piezometric level for the injection zone or leak must be assumed, leading to an assumed H value, H_a. The real and sought H value, H_r, is the difference between the assumed H_a and a systematic error in assumed piezometric level, H_0

$$H_r = H_a - H_0 \qquad [5]$$

where H_0 is either positive or negative depending on whether the real piezometric level (PL) is above or below the assumed piezometric level (APL).

Ignoring what is the actual PL around the injection zone is frequent. When a MW is installed in an aquitard, the apparently static water level in the MW pipe may differ from the real PL due to a lengthy time lag effect (Chapuis 2005a, 2005b, 2006, 2009). In an aquifer, a poor MW installation (resulting in preferential leakage along the pipe) or some previous injection (or pumping) may modify the flow net in the aquifer and thus, the observed water level may differ from the real PL for the slug test (Chapuis 1988; Chapuis and Sabourin 1989; Chapuis and Chenaf 2002, 2003). Therefore, in field conditions, some APL is frequently used in Eqs (1-4). When the H_a values are used instead of the H_r values, Equations 1 and 2 become

$$Q_{soil} = kcH_r = kc(H_a - H_0) \qquad [6]$$

$$Q_{inj} = -A\frac{dH_r}{dt} = -A\frac{dH_a}{dt} \qquad [7]$$

since (dH_0/dt) = 0, and thus Q_{inj} is always correctly obtained, even when using H_a.

Then, Eq. (3) becomes

$$\frac{dH_r}{dt} = \frac{dH_a}{dt} = -\frac{ck}{A}H_r = -\frac{ck}{A}(H_a - H_0) \qquad [8]$$

whereas Eq. (4) becomes

$$\ln\left(\frac{H_{r1}}{H_{r2}}\right) = \ln\left(\frac{H_{a1} - H_0}{H_{a2} - H_0}\right) = -\frac{kc}{A}(t_1 - t_2) \qquad [9]$$

Plotting log (H_{r1}/H_{r2}) against t gives a straight line. However, the field data are plotted as log (H_{a1}/H_{a2}) versus t, which will give either a straight line if $H_0 = 0$ (i.e. APL = PL), or a curve if $H_0 \neq 0$ (Chapuis et al., 1981; Chapuis, 2001).

However, the mass conservation equation (Eq. 8) can be used to obtain both k and H_0. A plot with $x = -dH_r/dt = -dH_a/dt$ (water velocity) and y = mean H_a value during the time interval dt gives a straight line of slope p, Eq. (8) becoming

$$y = H_a = -\frac{A}{kc}\frac{dH_a}{dt} + H_0 = px + H_0 \qquad [10]$$

The k value is then obtained from the slope $p = A/kc$ (Schneebeli 1954). Also from Eq. (8), the term (dH_a/dt) becomes null when $H_r = 0$ (or $y = H_0$), a mere outcome of Darcy's law. Thus, plotting y versus x with Eq. (8) yields a straight line that intersects the y-axis at ($x = 0$, $y = H_0 \neq 0$), thus giving the value of the error, H_0, that was made in evaluating the PL and all H_a values for the test (Chapuis et al. 1981; Chapuis 2001). Note that this velocity graph method was retained in several standards (e.g., AFNOR 1992; CAN/BNQ 1988, 2008) for field variable-head tests in boreholes or monitoring wells.

2.4 Interpretation using the Velocity Graph Method

The plot of Eq. (10) for the falling-head test in the swimming pool appears in Fig. 3. The assumed PL for the test was assumed to be at elevation zero. Note that this assumption does not influence the accuracy of the plot, because it is a constant systematic error. As a result, the mean H_a value during a time interval dt was equal to the mean water elevation in the pool during dt.

The velocity graph appears as a straight line as predicted by theory. The straight line intersects the null velocity axis close to H_0 = 45.0 m. This means that the real PL for the test was about 45.0 m. Since the flow rate could be easily evacuated through the pervious crushed stone in the trench, most head losses occurred through the defects or cracks in the pipes or connections. Physically, water in the pipes had a total head close to that of water in the pool, whereas water reaching the crushed stone was essentially at atmospheric pressure and thus had a total head equal to its elevation head.

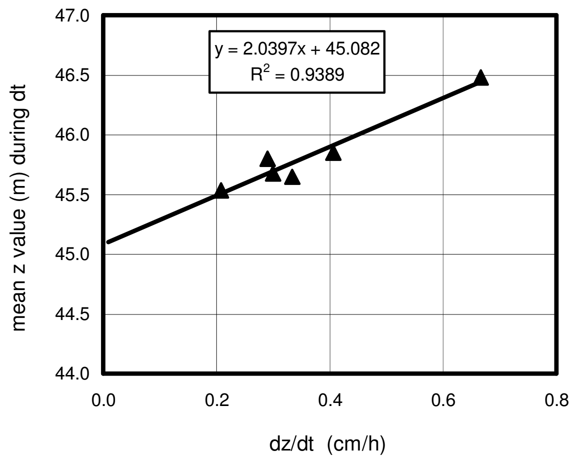

Figure 3. Velocity graph of the falling-head test using the swimming pool.

Thus, the velocity graph (Fig. 3) indicated that leakage occurred at an elevation close to 45.0 m. This was important since it meant that the bottom drainage pipe was not leaking (see Fig. 1) in its deep portion, but that at least one shallow pipe was leaking. The four pipes returning to the technical building had elevations between 44.91 and 45.13 m in three trenches. Excavation revealed four major leaks, and several minor ones. Several pipe connections that were initially glued together were found to be separated. In addition, the pipes had no longer their initial construction slope, and therefore could not be correctly drained before winter. As a result, water was trapped in certain sagging portions of the pipes. Since the pipes were about 1.1 m below the surface concrete slab, and since the frost penetration in the Montreal area can reach 1.5 m in most soils, but can go deeper in dry soils such as crushed stone, and can exceed 2.0 m below snow-cleared surfaces such as roads, the trapped water freezing in the pipes was first suspected; expansion could have damaged the pipes and caused the leaks.

3. EXCAVATION AND FINDINGS

However, sagging portions of the buried pipes clearly indicated some settlement issue. Several factors could explain differential settlement of the pipes: static weight of backfill, static weight of material stored on the ground surface, dynamic weight of circulating trucks during construction, freeze-thaw effects in the soil below the trenches, and potential internal erosion of the 0-20 mm crushed stone due to water leaks.

During the excavation, the dry density of the crushed stone was measured. Three modified Proctor Tests were performed and the optimum (OPM) was used as the reference for field data. In most contracts dealing with buried pipes, the usual reference is a minimum compaction of either 90% or 95% OPM. For this project, the trenches revealed that the 0-20 mm crushed stone in the trenches had not been compacted during construction, most dry densities measured values ranging between 65% and 80% OPM (Fig. 4). Note that the minimum dry density, as determined in laboratory, corresponded to 65% OPM. Only the 30-cm layer below the reinforced concrete slab around the pool was found to have been compacted at an average 93% OPM.

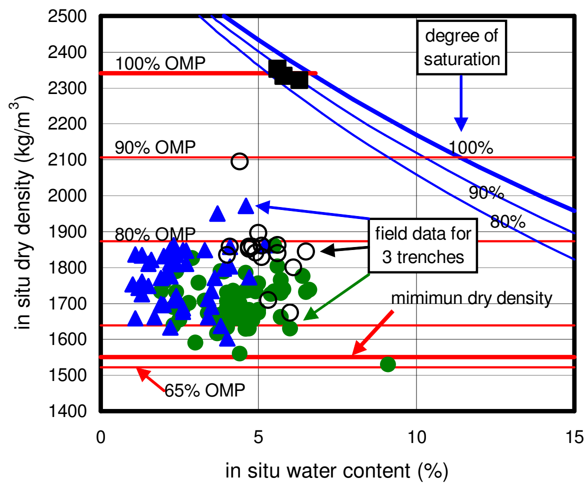

Figure 4. In situ dry density of the 0-20 mm crushed stone.

Therefore, the lack of compaction was a major problem. In the trenches, the pipes rested on a 60-cm layer of crushed stone. To illustrate how much this layer could have settled, an increase in its dry density from 66% to 80% OPM would yield a settlement of 107 mm.

The freeze-thaw effects in the underlying soils were not found to be an issue, because the soils were not frost sensitive and the water table had a low position. Therefore, the crushed stone had settled due to the lack of initial compaction, and the damage to the pipes and connections could also be related to this settlement. Large immediate settlement would have been due to static and dynamic loads.

In addition to large immediate settlement, some delayed settlement could have been due to an internal erosion of the 0-20 mm crushed stone. During excavation, it was noted that the crushed stone had lost its fine particles in the vicinity of the water leaks, the crushed stone being unable to retain its fine fraction. According to the grain size analyses of unwashed crushed stone (Fig. 5), and according to the usual criteria for internal erosion (Kezdi 1969; Sherard 1979; Kenney and Lau 1985), the gradation was unstable. This means that the final portion of the grain size distribution curve was too flat (Chapuis 1992; Chapuis et al. 1996; Chapuis and Tournier 2006) and consequently, either seepage forces or vibrations could produce segregation and displacement of fine particles for this gradation.

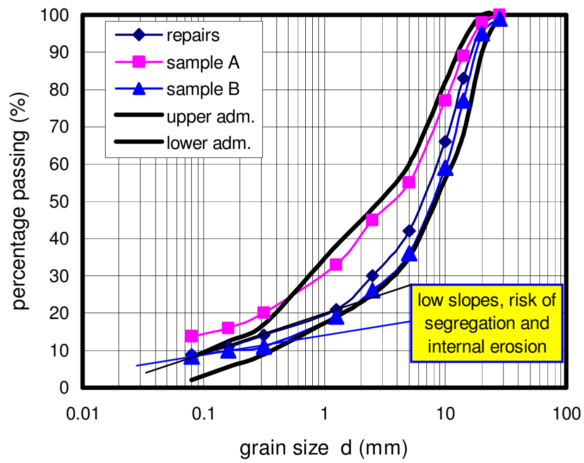

Figure 5. Grain size analyses of the 0-20 mm crushed stone.

CONCLUSION

A full scale falling-head permeability test was performed with the swimming pool as the variable-head reservoir. The data of this large-scale test were used to plot the water level downward velocity during a time interval versus the mean water elevation during this time interval. It was shown how the resulting velocity graph was used to determine the position of leaks. Excavations confirmed this position by finding broken pipes and connections. The damage and resulting leaks were due to high differential settlement caused by poor compaction of pipe foundations.

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