A LEACHATE RECOVERY SYSTEM AT THE PREMIER STREET LANDFILL, NORTH VANCOUVER, BRITISH COLUMBIA

by
Suzanne Szojka
Golder Associates, Vancouver, B.C.
James Atwater
University of British Columbia, Vancouver, B.C.
Richard Guiton
Golder Associates, Vancouver, B.C.

This paper presents a case history involving the containment and control of leachate emanating from the Premier Street Landfill in North Vancouver.

The Premier Street Landfill is situated along the eastern edge of Lynn Creek, above the Trans-Canada Highway. At the present time landfill leachate migrates with the ground water flow regime from the south-western end of the landfill, through permeable fluvial sediments, and enters Lynn Creek over a distance of approximately 100 m downstream of the landfill. Previous efforts to control the migration of landfill leachate in other locations at the landfill have included the construction of slurry walls and the establishment of a drain and leachate pumping system.

A leachate recovery system at the Premier Street Landfill has been designed with the objective of preventing the migration of leachate into Lynn Creek from the south-west corner of the landfill. The development of this recovery system was divided into three phases. An initial investigation involved the installation of one recovery well and four observation wells. During this first stage, geological and hydrogeological information on the area was collected and synthesized. A preliminary pump test was performed to assess the potential yield of the well and its influence on the flow system. Data from this pump test indicated the need for at least one additional recovery well and additional observation wells.

Upon completion of drilling, the two recovery wells were pumped simultaneously for a 24 hour period. This second pump test assessed the effect that operating both recovery wells would have on the flow system and resultant migration of landfill leachate.

Results from the pump test show that the recovery wells are successful in capturing the majority of flow emanating from the south-west corner of the landfill. Dropping water levels in all the observation wells and both recovery wells, the presence of a groundwater mound south of the pumping wells, and the existence of steep gradients towards Lynn Creek at the conclusion of the pump test, indicated that the ground water flow regime was still changing with time (transient) after 24 hours of pumping. However, with the achievement of steady-state flow conditions, under long-term pumping, these transient flow phenomena will disappear.

The nature of the flow regime and the efficiency of this leachate recovery system will be closely monitored during the long-term monitoring program.

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1.0 INTRODUCTION

The Premier Street Sanitary Landfill is situated in North Vancouver, B.C. along the eastern edge of Lynn Creek, approximately 1.5 km north of the Second Narrows Bridge, Figure 1. The site is bounded on the west by Lynn Creek and on the east by a steep glacial till escarpment. Surficial sediments in this area consist of fluvially derived coarse sands, gravels, cobbles and occasional boulders which overlie a dense gray silty-sand and gravel till.

At the present time, leachate originating from the older part of the landfill discharges into Lynn Creek at the south end of the landfill. The mixing of this landfill leachate, which contains high concentrations of total dissolved iron and manganese, with the more highly oxygenated Lynn Creek surface water, has resulted in visible staining of the creek bed. In order to prevent the migration of landfill leachate into Lynn Creek, a leachate recovery system has been designed that is currently undergoing final adjustments and testing.

2.0 SITE HISTORY

Land filling operations at the Premier Street Landfill have occurred in two distinct stages. The southern portion of the site was filled first and is termed the "old" landfill. In 1984, this southern site was nearing capacity and the landfill was expanded to encompass the adjacent area to the north, Figure 1. The northern extension of the landfill was closed to municipal refuse in 1988 and currently is being converted to park.

The "old" landfill, now covered and partially developed into a multi-use park, consists of fill up to 25 m high. It was developed by constructing a 6 m high dyke of loose silty sand and gravel along the east bank of Lynn Creek, and placing low permeability mineral fill at the base of the landfill, above the fluvial deposits. Filling took place by alternately layering garbage with layers of mineral fill. The land filling operations in the "old" landfill have resulted in a change in the topography such that ground surface elevations are greater than 60 m above sea level at the northern end of the landfill (where filling has been the greatest) and lower than 20 m above sea level in the southwestern corner.

A bentonitic slurry wall was initially constructed around the newer, more northerly extension of the landfill prior to refuse placement. Later, to arrest the seepage of leachate from the "old" landfill into Lynn Creek, this bentonite cut-off wall was extended southward between the bank of the creek and the western edge of the landfill (Figure 2), as a first phase of containment of leachate emanating from the "old" landfill. (The slurry wall supplements an interceptor drain originally installed in the mid 1970's. Water collected in this interceptor system is currently discharged to the municipal sanitary sewer from the south west corner of the landfill.) At the completion of this phase of slurry wall construction, groundwater flow from the south end of the "old" landfill was still occurring. The work described in this paper is that required to complete the leachate containment works at the Premier Street Landfill.

3.0 SITE HYDROGEOLOGY

Prior to the construction of the landfill, this area was a ground water discharge zone. Ground water migrated down-valley through permeable river sediments and the underlying low permeability glacial till and discharged into Lynn Creek. Hydraulic gradients measured in piezometers situated around the landfill still show a down-valley gradient in both the unconsolidated river deposits and the dense underlying till, and an upward gradient from the till into the fluvial sediments at the margins of the landfill. However, development of the area as a landfill has significantly altered the ground water flow regime. Down-valley ground water seepage through fluvial sediments north of the "old" landfill is now restricted by the impermeable bentonitic slurry wall constructed between the "old" landfill and the newer northern landfill extension. Ground water entering the "old" landfill from the east is also considered to be minimal due to the presence of the glacial till escarpment. The limited ground water seepage from the north and from the east would suggest that, currently, the majority of leachate is generated as a result of precipitation infiltrating through the landfill becoming contaminated as it percolates downward through the fill. Currently, leachate migrates along the bentonite cut-off wall at the western edge of the "old" landfill and discharges downstream of the cut-off wall to Lynn Creek.

Figure 3 is a contour map of the water table, measured December 14, 1989, in the area immediately to the south of the "old" landfill. The contours in Figure 3 represent lines of equal hydraulic potential. Perpendicular to these equipotential lines, flowlines have been sketched in showing the direction of groundwater flow. Ground water migrates toward Lynn Creek with the steepest gradients occurring at the edge of the existing slurry wall. This plot also shows ground water just below the playing field moving toward the current leachate drainage system.

3.1 GROUND WATER QUALITY

Ground water impacted by landfill leachate is characterized by conductivity values on the order of 1200 to 2000 *u*s/cm, as well as high chloride (72 to 152 mg/L), potassium (31.3 to 44.2 mg/L), total iron (31.8 to 155 mg/L), manganese (2.4 to 6.2 mg/L) and ammonia nitrogen (23 to 42.5 mg/L) concentrations. Leachate impacted ground water also shows moderate hardness due to the increased dissolved metals (calcium and magnesium) present. Bicarbonate concentrations are also elevated, when compared with relatively uncontaminated "background" water.

A number of options were considered for reducing groundwater discharge to Lynn Creek at the southern end of the landfill. The presence of a deep infilled channel in this area increased the cost of various remediation options considerably. It was decided to proceed with a pump and recovery system which, while not as certain a containment scheme as a cut-off wall, was considerably less expensive.

4.0 LEACHATE RECOVERY SYSTEM

The leachate recovery system is intended to prevent the migration of landfill leachate into Lynn Creek from the southwest corner of the landfill. It is anticipated that sufficient leachate will be captured to eliminate the adverse impact on the creek bed. The project was set out in the following three phases:

Phase 1: Investigation and Testing (including all drilling, pump testing and water chemistry to date)

Phase 2: Design and Construction of Approved System (involving the design of the overall system). This phase incorporates observation wells and the recovery well (P.W.1) completed during Phase 1). Phase 2 is ongoing.

Phase 3: Long Term Water Quality and Performance Monitoring

Phase 1 has included all drilling, pump testing and water chemistry to date. Phase 2, currently underway, has involved the design of the overall system which has incorporated wells completed during Phase 1.

During the initial Phase 1 work, four 8" diameter boreholes were drilled (BH 89-1 to BH 89-4). The location of these boreholes is shown on Figure 3. These holes were drilled to fill in gaps between an existing set of monitoring wells which were installed as part of earlier investigations. Multiple 2" diameter piezometers were installed in each of the boreholes, with the deepest piezometer located at the base of the aquifer, at the contact with the till surface. Each piezometer consists of a 3.3 m (10 ft.) length of screen, sand-packed along its length with a clean silica sand. Above the screen and sand-pack, a bentonite seal was placed in order to isolate each piezometer from adjacent piezometers, and allow the sampling of organic and inorganic constituents from discrete horizons within the aquifer.

In addition to the 4 observation wells, one 8" diameter recovery well (P.W. 1) was installed during Phase 1. Casing was initially driven to a depth of 10.6 m (35 ft.), the screen assembly (including a 0.91 m (3 ft.) tailpipe, 1.52 m (5 ft.), 150 slot stainless steel screen and 0.15 m (6") K packer), was then lowered to the bottom of the hole, and the casing pulled back to expose the assembly. The well screen was positioned over an interval from 9.29 m (30' 7") to 7.92 m (26'), directly above the till surface in the most permeable sediments.

The first recovery well (P.W.1) was pump tested October 23rd and 24th, 1989. The objective of this first pump test was to assess the potential yield of the well and determine its influence on the flow system. P.W.1 was initially pumped for 6 hours at 30 USgpm and then at 74 USgpm for another 6 hours. Results of this first pump test indicated the need for at least one additional recovery well, and 6 additional observation wells in order to more accurately determine the affect of pumping on the flow system, especially in the vicinity of the cut-off wall. Four of these six observation wells (BH 89-5,6,7 and 10), consisting of a single 1 3/4" standpipe inside a 6" diameter borehole were installed for hydraulic control only. The remaining two observation wells (BH 89-8 and BH 89-9) were constructed in the same manner as those in Phase 1 and are shown on Figure 3.

The second recovery well (P.W.2) was positioned at a distance of 66 m from P.W.1. This spacing was based on the radius of influence observed during the pump test of P.W.1, anticipated well interference effects and consideration of aquifer heterogeneity and thickness. P.W.2 was screened over an interval from 8.53 m (28 ft.) to 10.05 m (33 ft.). As with the first recovery well (P.W.1), P.W.2 was also constructed with a 0.91 m (3 ft.) tailpipe and a 1.52 m (5 ft.) stainless steel screen. The location of the second recovery well is also shown on Figure 3.

Stratigraphy was logged during drilling, providing a more detailed understanding of the thickness and nature of surficial deposits in this area. The depth to the till surface varied from 7.78 m (25 1/2 ft.) at borehole 89-3 to over 15.2 m (50 ft.) at borehole 89-8. With the exception of sediments logged at borehole 89-8, surficial sediments situated above the till consist primarily of gray gravels, cobbles and boulders with up to 20% coarse sand and up to 10% fine sand and 10% silt. These sediments are overlain near the ground surface by varying thicknesses of fill and forest litter. The fill, composed of loose dark brown coarse sands, fine gravels and boulders and occasional waste materials such as tires and logs, thickens towards Lynn Creek. A maximum thickness of 4.72 m (15-1/2 ft.) of fill was observed at borehole 89-5.

At borehole 89-8, the stratigraphy was different from that observed in the other boreholes. Below the layer of coarse gray gravels, cobbles and boulders present across the rest of the site, sand and silt, interbedded with layers of coarse sands and gravels, were encountered. Below these sediments, at a depth of 13.10 m, a layer of silt was encountered. Review of the borehole log for monitoring well W2, approximately 40 m away, suggests that this silt layer may be continuous and indicative of an old, infilled, river channel.

Upon completion of the drilling and well development program, the two recovery wells were pumped simultaneously for a 24 hour period. During this period water levels in 28 piezometers (including piezometers installed in 1985 and 1987, as well as those constructed for this leachate recovery system) were monitored. The purpose of this second pump test was to assess the effect that both pumping wells would have on the flow system and resultant migration of landfill leachate. Results from this second test are representative of flow conditions that would arise during operation of the recovery system and, therefore are discussed in detail below.

4.2 DISCUSSION OF PUMP TEST RESULTS

The pump test ran for 24 hours. P.W.1 was pumped at 88 USgpm and P.W.2 at 45 USgpm, making the total discharge 133 USgpm. The water pumped during this test was discharged to the sanitary sewer at the Pumping Station. Figure 4 is a contour plot of the water table after 22 hours of pumping. As can be seen from the figure, pumping of both recovery wells creates a steep cone of depression such that ground water flowing towards the southwest, from the corner of the landfill where the current municipal yard is located, is captured by P.W.2. Leachate originating from the waste situated below the playing fields is also captured by the recovery wells, as is the majority of leachate which is flowing along the existing slurry wall. Figure 4 is the most conservative plot (i.e., the worst case scenario) of pump test data and shows that a very small amount of leachate may still have been migrating in a narrow zone along the slurry wall at the conclusion of the pump test.

Dropping water levels in all the observation wells and both recovery wells, the presence of a groundwater mound south of the pumping wells, and the existence of steep gradients toward Lynn Creek at the conclusion of 24 hours of pumping, are indicative of a ground water flow regime which is still changing with time (i.e. transient). Once steady-state flow is achieved a state of dynamic equilibrium will exist where the flux of water delivered to the recovery system will equal the amount of water necessary to maintain the position of the water table at every point. When this occurs, the water levels in both the observation and pumping wells will stabilize. Under long-term pumping of the recovery wells, steady-state conditions will be achieved.

The flow of ground water southwest of the ground water divide toward Lynn Creek resulted from the drainage of ground water present prior to the start of the test and not from leachate which had by-passed the recovery wells during the pump test. With the achievement of steady-state flow conditions, under long-term pumping, this drainage will be completed since ground water inflow to this area will be greatly reduced, and the gradients toward Lynn Creek will flatten.

Figure 4 shows that P.W.1 and P.W.2 are successful in capturing the majority of the flow emanating from the landfill. The amount of leachate which <u>may</u> have been migrating along the edge of the slurry wall and by-passing the recovery wells is small, and will likely be captured once long-term pumping is underway and steady-state flow conditions are achieved.

The nature of the flow regime in the vicinity of the slurry wall will be closely monitored during the Phase 3 long-term monitoring program to determine whether or not leachate is by-passing the recovery wells. Should it be determined during the long-term monitoring program that migration of leachate along the slurry wall is occurring, and if the amount is deemed to be significant enough to cause staining along the creek, installation of a third recovery well may be considered.

5.0 PROPOSED SYSTEM

The requirements of the leachate recovery system and a brief discussion of the proposed system are presented below. Design of this proposed system is currently underway.

5.1 SYSTEM REQUIREMENTS

The most effective operation of the leachate recovery system involves satisfying the three following requirements:

1. The system will respond to fluctuating ground water levels. Pump test results indicate that the groundwater levels are heavily influenced by precipitation and storm events.

2. The operation and maintenance requirements of the final leachate recovery system will be minimal.

3. Long-term energy requirements will be minimized.

5.2 SYSTEM DESIGN

The proposed pumping system design consists of two production wells with submersible pumps shrouded and placed in the tail-pipes to increase the amount of maximum drawdown available. The pumps will run continuously and will be fitted with variable speed controllers tied to a relatively narrow depth band (approximately 0.3 metres - guess at this time). Once the system is operational; the thickness of band and the optimum seasonal drawdown will be established. Each well will be fitted with electrodes for the speed controllers and duel pressure transducers, for monitoring, drawdown adjustments and alarms. Discharge will be to the sanitary sewer with a flow recording device.

6.0 SUMMARY

A groundwater pumping system is currently being designed to control the migration of leachate from the south-end of the Premier Street landfill. Observation and production wells have been installed and tested. The design of the control system is in its final stages and it is anticipated that the system will be operational by summer.

The system will require monitoring to first determine optimal operating conditions and, thereafter, to ascertain whether or not the system captures enough leachate to eliminate the current impact on Lynn Creek.

REFERENCES

Golder Associates, 1976. "Report to District of North Vancouver on Seepage Control Slides, Premier Street Sanitary Landfill". Report No. V76051.

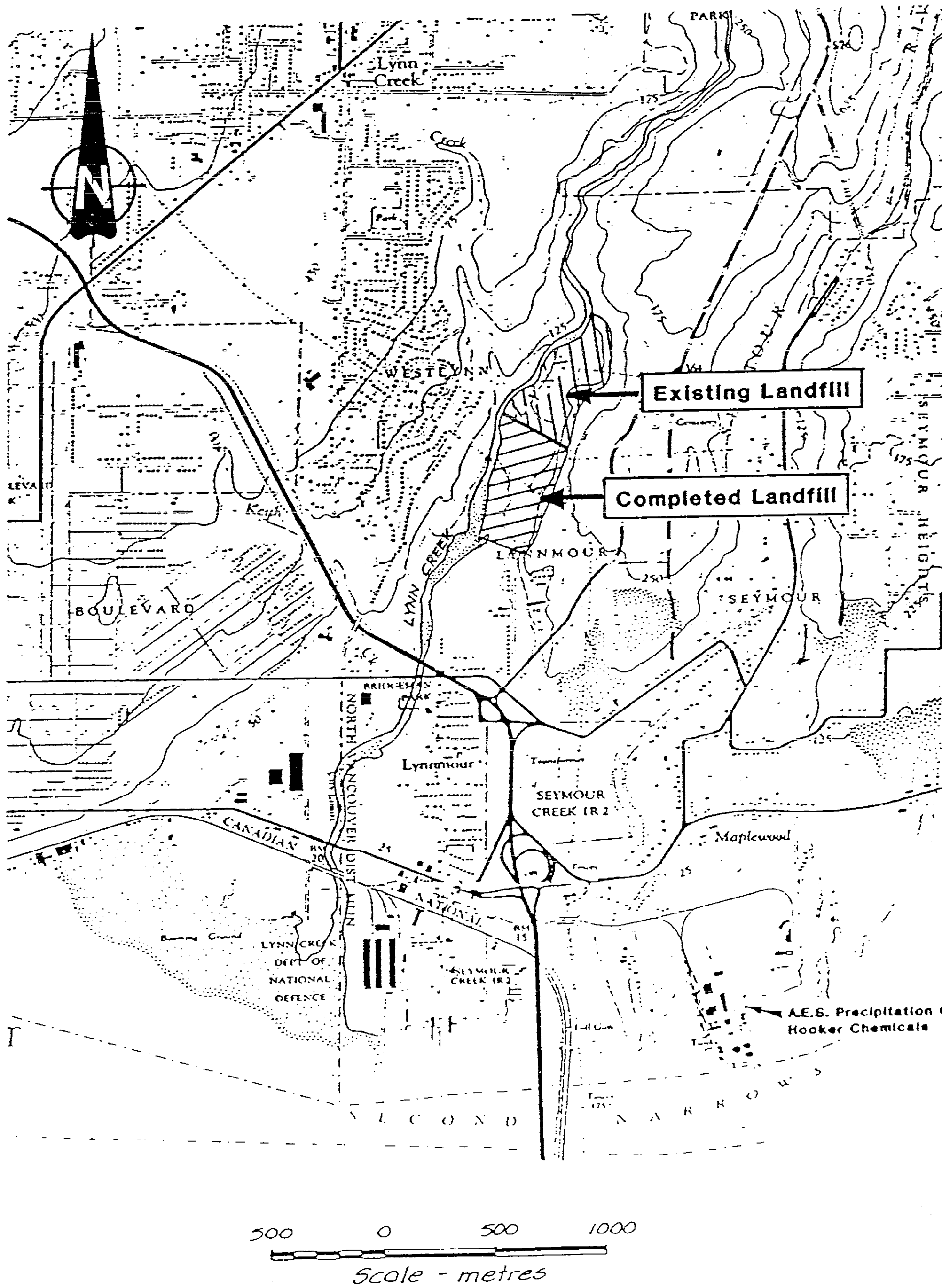
Golder Associates, 1982. "Letter Report to District of North Vancouver on Leachate Seepage, Premier Street Landfill". Report No. 822-1034.

Golder Associates, 1983. "Report to Associated Engineering Services Ltd. on Hydrogeological Investigation, Premier Street Landfill." Report 832-1545.

Hvorslev, M.J., 1951. "Time Lag and Soil Permeability in Ground Water Observations". Bulletin No. 36, Waterways Experiment Station, Corps. of Engineers, U.S. Army, Vicksburg, pp. 50.

LOCATION OF SITE

Figure 1



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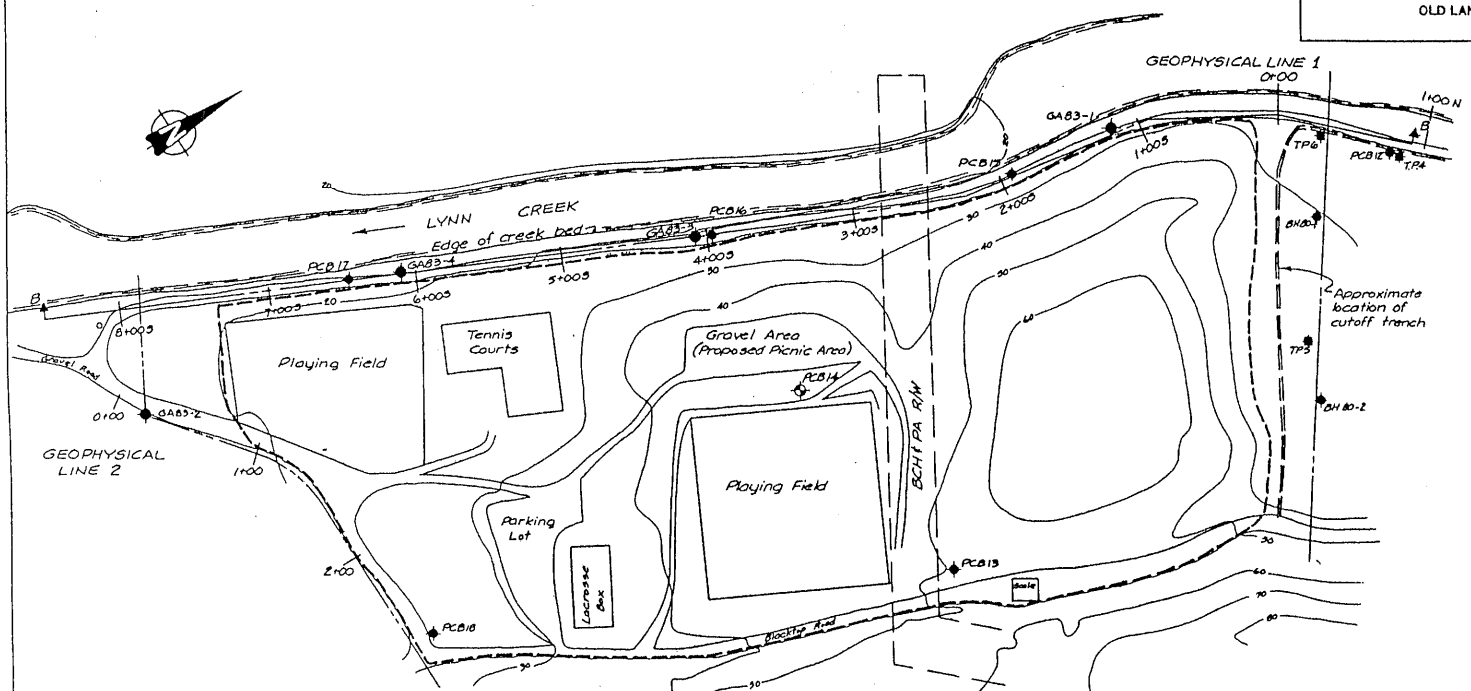
OLD LANDFILL

Figure 2

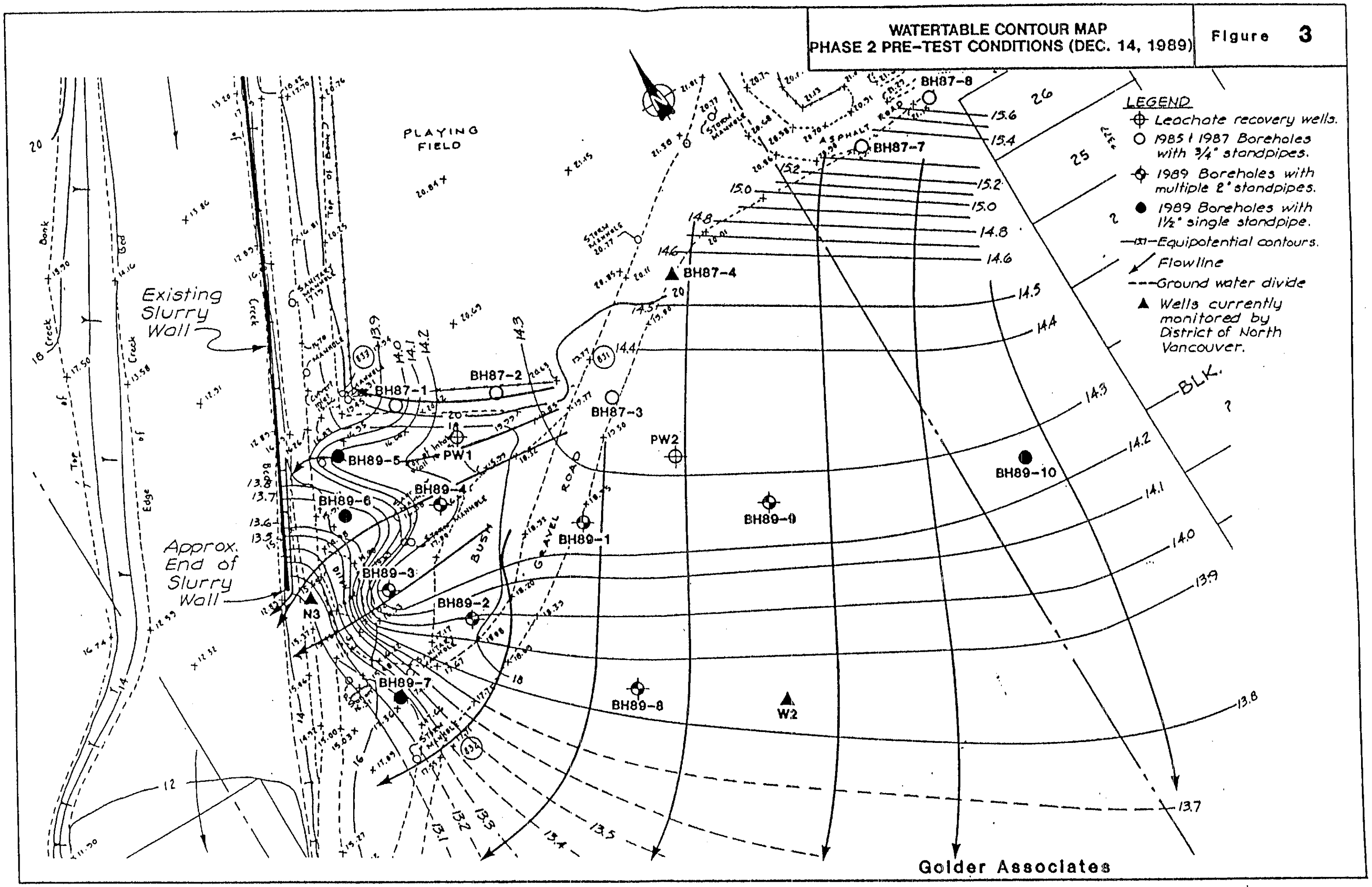


LEGEND
- Boreholes - Golder Associates 1983 investigation.
- Boreholes - previous investigations.
- Test Pits - previous investigations.
- Outline of completed fill

REFERENCE
Topographic Survey of parts of District lots 851, 1620, 614, 613, 612, 615 by Looch Hobbs & Kyler, B.C. Land Surveyors; scale 1:1000.

Drawn R.D.
Reviewed
Date: Aug. '83

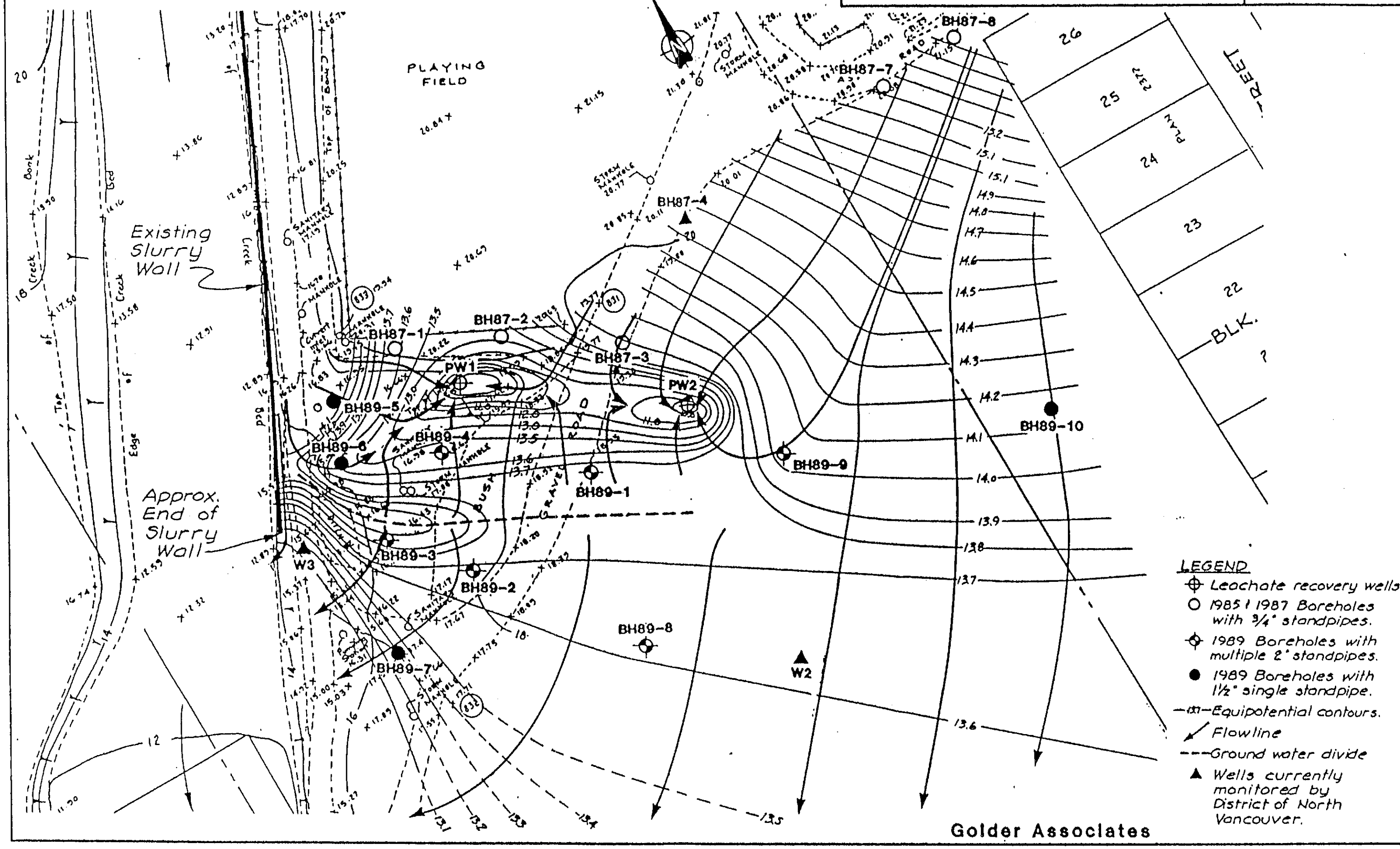
Golder Associates



WATERTABLE CONTOUR MAP
PHASE 2 PRE-TEST CONDITIONS (DEC. 14, 1989)
Figure 3
LEGEND
Leachate recovery wells.
1985 & 1987 Boreholes with 3/4" standpipes.
1989 Boreholes with multiple 2" standpipes.
1989 Boreholes with 1½" single standpipe.
Equipotential contours.
Flowline
Ground water divide
Wells currently monitored by District of North Vancouver.
PLAYING FIELD
Existing Slurry Wall
Approx. End of Slurry Wall
BH87-1
BH87-2
BH87-3
BH87-4
BH87-7
BH87-8
BH89-1
BH89-2
BH89-3
BH89-4
BH89-5
BH89-6
BH89-7
BH89-8
BH89-9
BH89-10
PW1
PW2
N3
W2
ASPHALT ROAD
GRAVEL ROAD
BUSH
Creek
BLK.
Golder Associates

WATERTABLE CONTOUR MAP DURING PHASE 2 PUMP TEST (22 HRS.)

Figure 4



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