HYDRAULIC CHARACTERISTICS OF MUNICIPAL REFUSE

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ABSTRACT: A review is made of the hydraulic conductivity and other hydraulic parameters of municipal waste. The hydraulic conductivity can be assessed indirectly from measured field parameters and water balance. A test well penetrating about 100 ft of refuse was installed and pumped for about one day at 20 gpm and 2-1/2 days at 12 gpm. Drawdowns were measured at three observation wells and the pumped well. Difficulties were encountered during drilling and subsequent monitoring of leachate levels. The hydrogeologic parameters were computed using conventional hydrogeologic analysis. The results of a pumping test of leachate from a municipal landfill are presented. Based on the results of the pumping test, it is concluded that hydraulic conductivity of municipal refuse is about 10^{-3} cm/s. It is also concluded that pumping of leachate from a municipal landfill is feasible for control of leachate release to ground water.

INTRODUCTION

The purpose of this paper is to present an overview of the hydraulic properties of saturated refuse and describe a recent leachate pumping test on a well completed within a solid waste municipal landfill. New solid waste facilities are required to install liners and in situ leachate control and collection systems. Proper assessment of the hydraulic characteristics of refuse is an important design element because of the potential impacts related to uncontrolled migration of leachate on ground-water quality. Fig. 1 shows a schematic design of a modern land disposal facility.

Surface water and atmospheric models together with hydraulic flow models are used to assess the percolation of water into and through the waste (Schroeder et al. 1983; Fenn et al. 1975). One objective of landfill design is to limit the hydraulic head above the liner to reduce the possibility of the accumulation and subsequent migration of leachate.

At uncontrolled landfills, a leachate mound within the refuse develops until the newly established flow system reaches a state of dynamic equilibrium with the existing hydrogeologic conditions. The leachate mound stabilizes when the recharge rate of water entering the landfill approximately equals the discharge of leachate leaving the system. Variables affecting the distribution of hydraulic head within the refuse and underlying materials include total precipitation rates, field capacity of the refuse, vertical and horizontal hydraulic conductivity of the refuse, and hydraulic properties of the underlying hydrogeologic units. The geometry of a leachate mound can be directly measured in the field, or it can be predicted by knowing the size of

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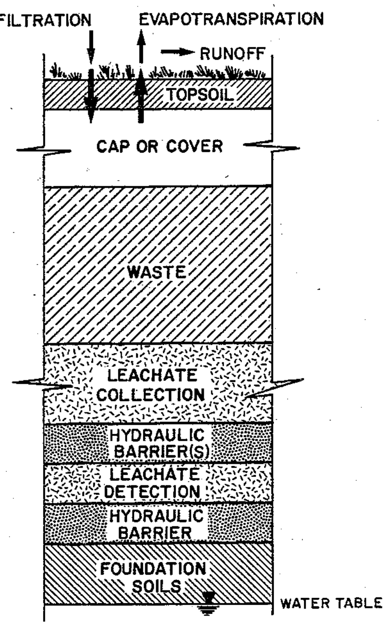

FIG. 1. Schematic Design: Landfill Disposal Facility

the landfill and estimating these variables (Oweis and Khera 1986).

Leachate generation and discharge at an uncontrolled landfill continues until a disequilibrium to this balance is created. The current practice of installing a low-permeability cap on the landfill surface after closure reduces recharge through the refuse (if the cap remains uncracked). Discharge of leachate is typically managed by installing a perimeter leachate collection drain and/or a vertical barrier wall, as illustrated in Fig. 2.

At an uncontrolled landfill site, the hydraulic characteristics of the refuse and the site hydrogeology are key elements in the design of an effective leachate collection system. They directly affect the cost of construction and the cost of leachate treatment. Pumping of leachate from wells within the landfill to control the discharge may offer advantages in terms of relatively quick removal of the leachate at a favorable cost benefit compared to other types of remediation. In the event of a failure of the leachate collection and removal (LCR) system beneath the bottom liner of a new landfill, pumping of leachate may be an attractive option. In addition, leachate removal helps reduce the weight of the refuse and the pore fluid pressure in the foundation soils supporting the fill. This leads to enhanced stability of an existing landfill on soft clay soils.

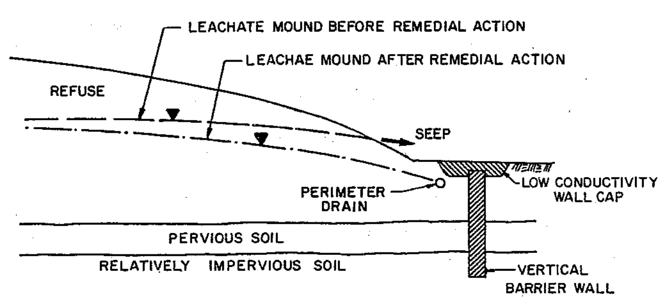

FIG. 2. Typical Site Remedial Action at Old Landfill

Pumping Test

A leachate pumping test was conducted at a municipal landfill in northern New Jersey. The purpose of the test was to investigate the feasibility of discharging leachate from an array of pumping wells within the landfill. The location of the test (Fig. 3) was selected to meet operational requirements because the landfill was active at the time of the test. Based on fluid measurements in wells installed for the study, the landfill contains a leachate mound with a maximum saturated thickness of about 35 ft (10.67 m). The average thickness of refuse at the location of the test is about 105 ft (32.0 m). The leachate pumped was collected and subsequently treated. The methods used for analyses of the data included the nonequilibrium formulas of Theis (1935) and Boulton (1963), along with the straight line solutions of Jacob (1946). These methods provide an estimate of transmissivity and specific yield, and show graphically the existence and effects of delayed yield drainage. Formulas and methods of solution for these techniques are common in the literature (Lohman 1972) and are not discussed in detail herein.

Under water-table conditions, gravity drainage of interstices reduces the saturated thickness and, therefore, the computed coefficient of transmissivity of the aquifer. Observed values of drawdown must be adjusted to compensate for the decrease in saturated thickness before the data can be used to compute the hydraulic properties of the aquifer. The Jacob equation (Driscoll 1986) was used to adjust the measured drawdown values:

$$s' = s - \left(\frac{s^2}{2m}\right) \qquad (1)$$

where s' = drawdown that would occur in an equivalent nonleaky artesian aquifer, in feet; s = observed drawdown under water-table conditions, in feet; and m = initial saturated thickness of aquifer, in feet.

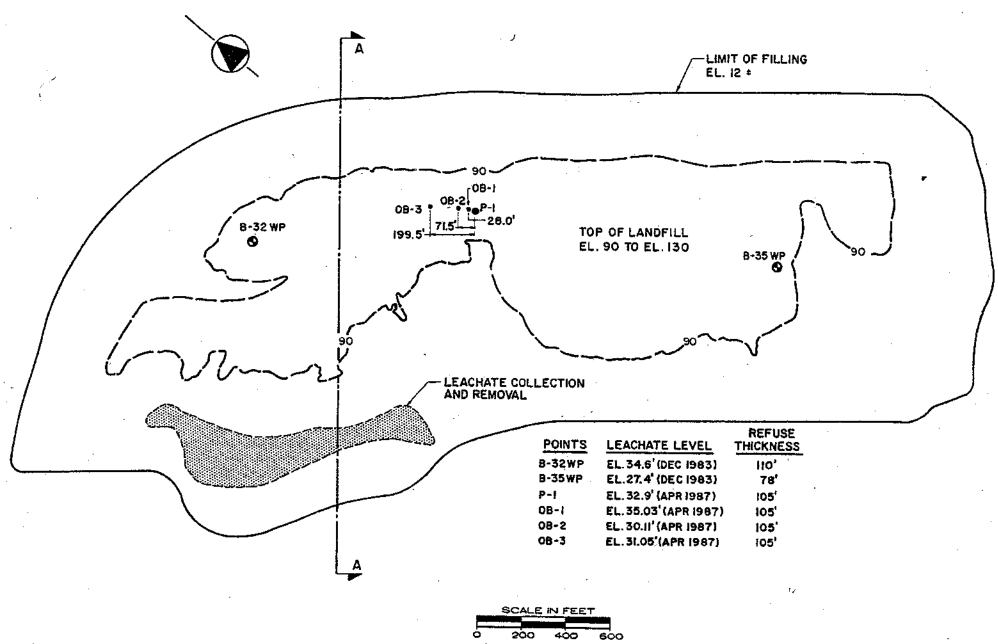

FIG. 3. Site Location Plan

Well Construction

The test well consisted of a 6-in. diameter stainless steel casing and screen assembly installed in a 20-in. (0.51 m) diameter hole as illustrated in Fig. 4. The use of the very thick gravel pack was intended to maximize hydraulic continuity with the aquifer and to offset anticipated plugging of the screen and gravel pack by paper, plastic, fibers, and other debris. Stainless steel 60-slot wire wrap screen was selected for the same reason.

Various alternatives were considered for drilling the 20-in. (0.51-m) hole. A bucket auger was discounted because past experience indicated a high probability of hole collapse below the leachate level. A pile driver was not used because driving 20-in. (0.51-m) or 16-in. (0.41-m) pipe through refuse was known from past experience to be difficult and costly. The cable tool method of well construction was finally selected.

Gas release from the hole was troublesome. The drillers added water to maintain a slurry at the base of the casing, and this controlled the situation. Organic vapor detectors (methane explosimeters) were used to monitor working conditions, especially during welding operations. Two 100-t jacks were used to pull back the outer casing as the gravel pack was introduced in about 5-ft (1.5-m) increments. The gravel pack consisted of 3/8-in. (9.5-mm) washed pea gravel.

Three observation wells, installed using hollow-stem auger equipment, were constructed of 2-in. (50.8-mm) diameter stainless steel casing and 90 ft (27.43 m) of 10-slot well screen. Observation wells OB-1, OB-2, and OB-3 were installed at distances of 28 ft (8.53 m), 71.5 ft (21.8 m), and 199.5 ft (60.82 m), respectively, from the test well (see Fig. 3). In addition, a 1-1/2-in.

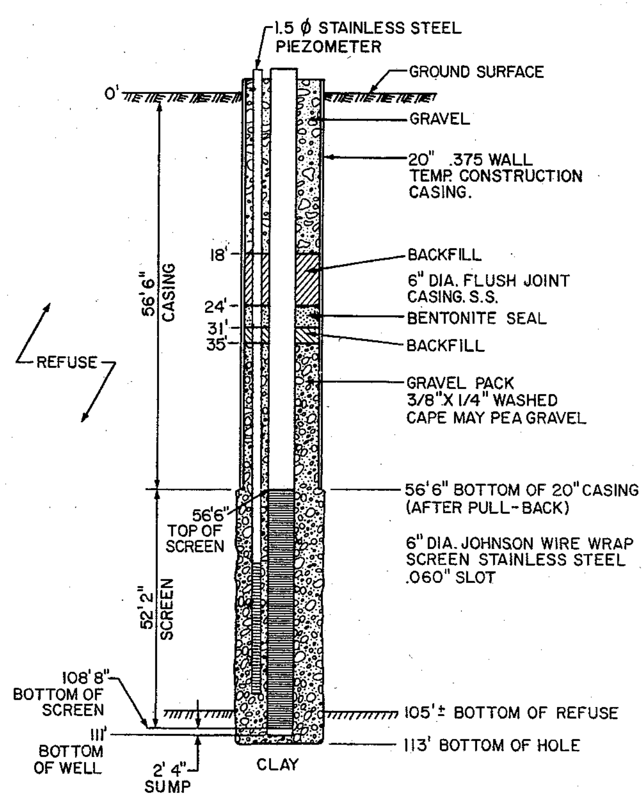

FIG. 4. Test Well: Detail Section

(38.1-mm) piezometer (P-1) was installed within the gravel pack of the test well to measure fluid levels immediately outside the well screen.

Fluid levels were measured at depths of 65–70 ft (19.8–21.3 m) below the landfill surface. High gas flow from the three observation wells, along with high fluid conductivity and foaming, caused difficulties with fluid level measurement. To resolve this problem, 3/4-in. (19-mm) high-temperature PVC pipe was installed on the bottom of each observation well screen. Special well caps were constructed to allow access for the monitoring equipment and to prevent free gas discharge at the surface. This made working in the area tolerable and prevented possible toxic and explosion hazards. In addition, sealing the well head reduced the amount of bubbling and foaming in the well. This reduced fluctuations and allowed more reliable fluid level measurements. A threaded nipple in the cap also allowed gas pressures to be measured periodically in each of the observation wells using a water manometer. Gas pressures ranged from 0.21 psi to 0.83 psi.

Temperatures in the observation wells were measured by lowering a thermometer to record the maximum temperature in each hole. Vapor temperatures in the observation wells ranged from 140° F to 150° F (60 to 65° C). Discharge water temperatures during the pumping tests were measured at 132° F (55.5° C). Use of a flow meter to measure discharge rates during pumping was aborted due to plugging by fibers when the pumping rate reached 45 gpm (2.83 L/s) during initial experimental pumping. Flow calculations were subsequently based on measured discharge temporarily diverted into a calibrated container over a known time interval.

Site Description

The landfill at which the pumping test was conducted is located in northern New Jersey. Fig. 3 shows the approximate toe and top of the landfill. At the time of the pumping test, elevations at the top of the landfill varied from about +90 to about +130. At the location of the test area on the top of the landfill the ground elevation varied from +96 to +99 ft (29.27 to 30.18 m). Data regarding the elevation of the leachate mound are shown on Fig. 3. The 1983 leachate level in the pump test area was probably higher than the 1987 level, because in 1985 an area in the southwest side of the landfill was excavated, an LCR system was installed, and the area subsequently filled with refuse. The approximate extent of the LCR system is shown in Fig. 5. The toe of the landfill is typically at +12 ft (3.66 m). The prevailing ground-water level outside the landfill is typically at +3 ft (0.9 m). Fig. 5 shows a subsurface section across the landfill. The refuse is typical municipal refuse, which includes household garbage and light industrial waste (paper, plastics, etc.). Based on indirect measurement (Oweis et al. 1986), an average unit weight of 43 pcf (6.8 kN/m^3) is inferred. The refuse typically extends to −6 ft (−1.83 m) (top of varved clay).

The thick varved clay has an average vertical laboratory hydraulic conductivity of about 10^{-7} cm/s. The horizontal conductivity is higher and may be about 10^{-6} cm/s. The surficial organic silt layer that prevails outside the landfill is either very thin or nonexistent beneath the landfill. It is theorized that the layer was either excavated or removed by "mudwaving" as dumping was in progress. The intervening silty sand layer increases in thickness towards the western portion of the landfill. The hydraulic conductivity of the

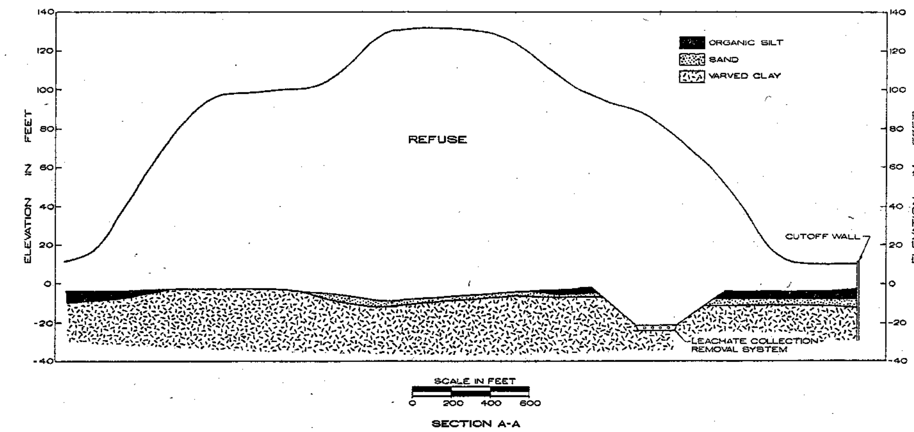

FIG. 5. Subsurface Section

silty sand is about 10^{-3} cm/s based on grain size data. It is either thin [less than 1 ft (0.3 m)] or missing over most of the landfill footprint. The side slopes of the landfill are typically at three horizontal to one vertical.

Leachate Generation and Hydraulic Conductivity

Water-balance calculations were made using program HELP (Schroeder et al. 1984). A synthetic 20-yr rainfall record generated for Central Park, New York, was used for the analysis. The 20-yr record with an average annual rainfall of 49.68 in. (1,261.9 mm) was generated by repeating the available 5-yr record four times. An effective hydraulic conductivity of 10^{-3} cm/s, porosity of 0.4, and an average refuse thickness of 120 ft (36.58 m) were assumed. The computed leachate generation was 30.5 in. (774.7 mm) for SCS curve number 87. Alternative calculations (Fenn et al. 1975) yield about 22-in. (558.8 mm) of leachate generation. For the aforementioned assumptions, HELP predicts a peak daily leachate head of 34.3 ft (10.46 m) assuming that the prevailing ground-water level is below the top of the clay layer. The actual leachate generation is probably less than 30 in. (762 mm)/yr and closer to 20 in. (308 mm)/yr because runoff along the 3H:1V slope is not accounted for in the HELP program. If the toe of the landfill is considered to be a drainage boundary, and assuming an average drainage distance of 1,200 ft, then the leachate buildup above the prevailing water level outside the landfill can be approximately estimated based on (Harr 1962)

$$h = \frac{l}{2}\sqrt{\frac{e}{k}} \qquad (2)$$

where h = maximum leachate buildup; e = average percolation; k = hydraulic conductivity of refuse; and l = drainage distance = 1,200 ft.

For e = 20 in. (508 mm)/yr, $k = 10^{-3}$ cm/s (12,415.7 in./yr), h is 24.1 ft (7.35 m) [i.e., el. +27.1 ft (8.26 m)]. For $k = 0.7 \times 10^{-3}$ cm/s, h is at about 31.8 ft (9.69 m), which is in accord with field observations. Piezometers installed in the varved clay indicated significant excess pore pressures due to the weight of the refuse. Piezometric elevations of 54.6 ft (16.64 m) and 43.8 ft (13.35 m) were measured in borings B-32 and B-35, respectively. The screen elevations were at −50 ft (−15.24 m) and −78 ft (23.78

TABLE 1. Hydraulic Parameters

Material (1)	Specific yield (% volume) (2)	Porosity (% volume) (3)	Field capacity (% volume) (4)	Initial moisture content by volume (5)
Clay	up to 10	>45	45–61	—
Sand	10–30	25–40	16–22	—
Sand and gravel	15–30	15–25	17	—
Municipal refuse	10	40–50	20–35	10–20

Note: Basis is Chow (1964), Schroeder et al. (1983), this paper, and Fenn et al. (1975).

TABLE 2. Summary of Determinations of Hydraulic Conductivities of Refuse

Source (1)	Unit weight (pcf) (2)	Hydraulic conductivity (cm/s) (3)	Method (4)
Oweis and Khera (1986)	41 (est.)	Order of 10^{-3}	Estimated based on field data
This paper	41	10^{-3}	Pumping tests
Koriatis et al. (1983)	55	5.12×10^{-3} to 3.15×10^{-3}	Laboratory tests
This paper	60–90 (est.)	1.5×10^{-4}	Falling head field test
This paper	40–60 (est.)	1.1×10^{-3}[a]	Test pit
Fungaroli et al. (1979)	7–26 (milled refuse)	10^{-3} to 2×10^{-2}	Lysimeter determination

[a]Infiltration rate.

m) at borings 32 and 35, respectively. The recharge to the refuse due to the pore pressure dissipation from the underlying clay stratum is not considered in these leachate generation models. This source, however, is expected to be insignificant due to the relatively low hydraulic conductivity of the clay.

Table 1 provides some hydraulic parameters for refuse compared to soils. The moisture content for refuse is defined based on a vol/vol basis, which differs from the usual geotechnical definition (weight of water/weight of solids).

The specific yield is the amount of water that a unit volume of unconfined saturated soil or refuse gives up by gravity drainage. The field capacity defines the upper limit of soil moisture available for plant use (wilting point is the lower limit). Physically, it is the water held after gravity drainage. Gravity drainage is not possible when the volumetric moisture content is less than the field capacity.

Table 2 provides published determinations of the hydraulic conductivity for refuse. It appears that in the absence of site-specific data, a saturated conductivity of 10^{-3} cm/s is a reasonable first estimate for typical municipal refuse that has good compaction.

Step-Drawdown Test

A step test was performed to assess the pumping rate at which the pumping test should be conducted. The time-drawdown data for the step test are shown in Fig. 6. The discharge and drawdown data for the step test are shown in Table 3. The relationship between Q and S is not linear, which suggests turbulent flow conditions. If the data point for the second step is not included, the relationship becomes essentially linear. In stepping to 30 gpm (1.89 L/s), some clogging of the filter and the screen may have occurred. Based on the step test, it was concluded that a 20-gpm (1.26 L/s) pumping rate could be maintained for 48 hr.

Constant Rate Pumping Test

Based on the results of the step test, a constant rate test was run with a selected pumping rate of 20 gpm (1.26 L/s). The drawdown time relationships at P-1, OB-1, and OB-2 are shown in Figs. 7, 8, and 9. At the end of 24 hr, the fluid level in the test well reached 100.09 ft (30.51 m) below the top of the casing [test drawdown of 28 ft (8.531 m)]. Soon afterwards, the fluid level in the well reached the pump intake and the test was termi-

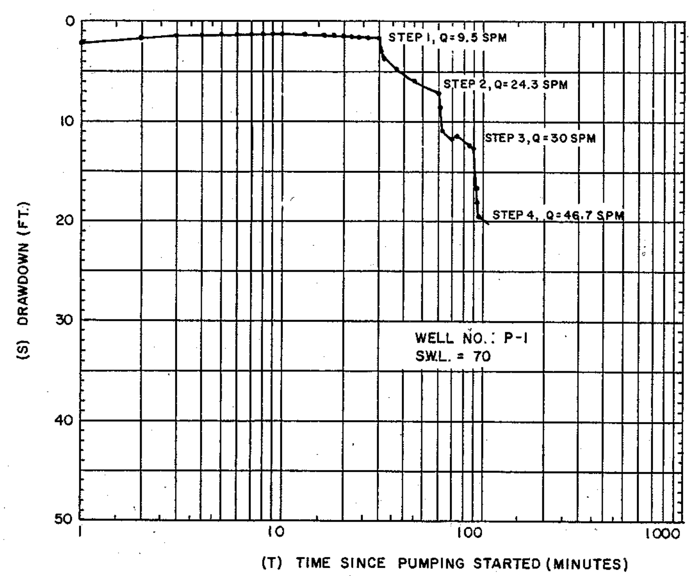

FIG. 6. Step Drawdown Test

TABLE 3. Discharge and Drawdown Data from Step-Drawdown Test

Step number (1)	Yield (Q) (gpm) (2)	Time of pumping (min) (3)	Total drawdown (S) P-1 (ft) (4)
1	9.5	30	1.78
2	24.3	30	7.25
3	30	30	12.89
4	46.7	30	21.64

Note: 1 ft = 0.305 m; 1 gpm = 0.003 L/s.

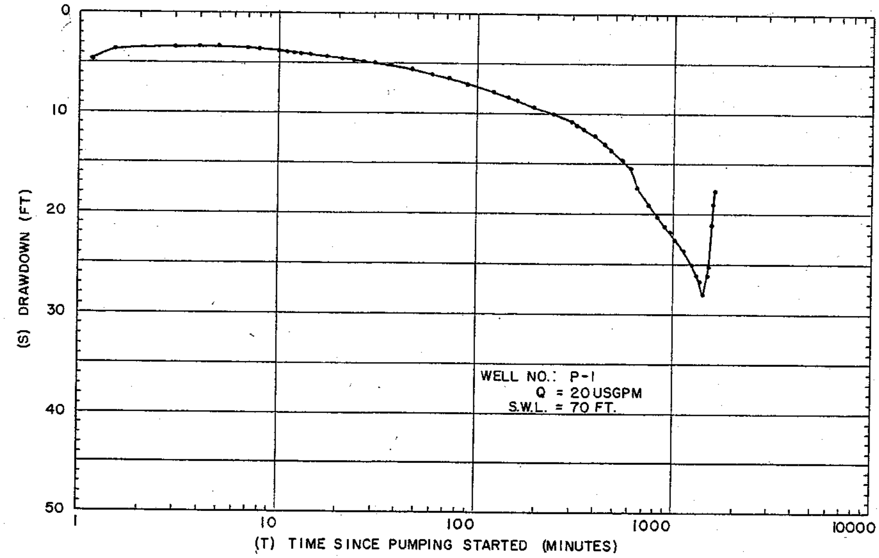

FIG. 7. Pumping Test Analysis: P-1, Q = 20 GPM

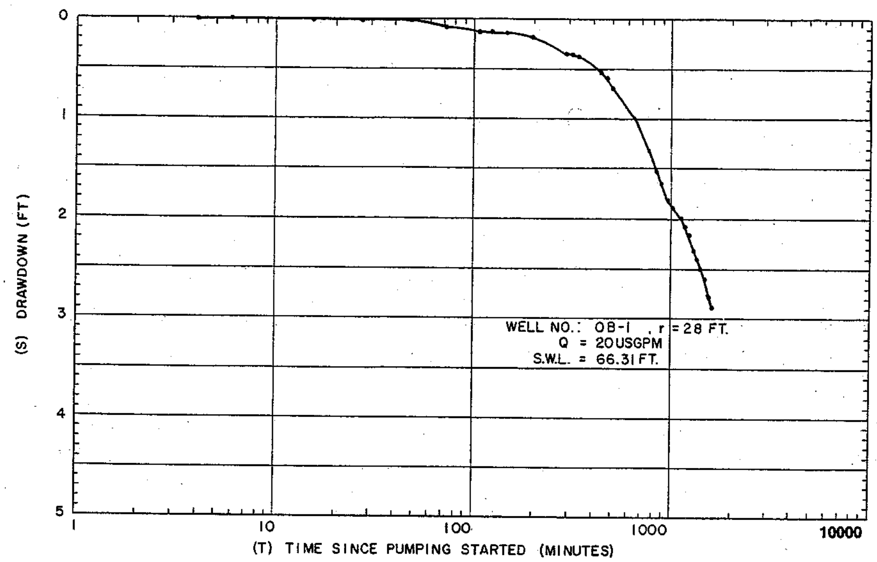

FIG. 8. Time-Drawdown at OB-1, Q = 20 GPM

nated at a total elapsed time of 1,640 min. Recovery was measured (Fig. 10) for 2.5 days, at the end of which the water level in the pumping well (P-1) was 1.64 ft (0.5 m) below the pretest static water level. The parameter T in Fig. 10 is the time since pumping started, and T' is the time since pumping ended.

A second pumping test was conducted at 12 gpm (0.76 L/s) for about 2.5 days (3,510 min) and was terminated before drawdown reached the pump intake [drawdown of 24.27 ft (7.4 m)]. Fig. 11 shows the time-drawdown curve for this pumping rate. Fig. 12 shows the recovery data.

Total drawdown in observation wells OB-1 and OB-2 amounted to 2.88 ft (0.88 m) and 1.59 ft (0.48 m), respectively, at the end of the 20-gpm (1.26-L/s) pumping rate. If the pretest (highest) static water levels are considered, then the maximum drawdowns are 4.22 ft (1.29 m) and 1.63 ft (0.5

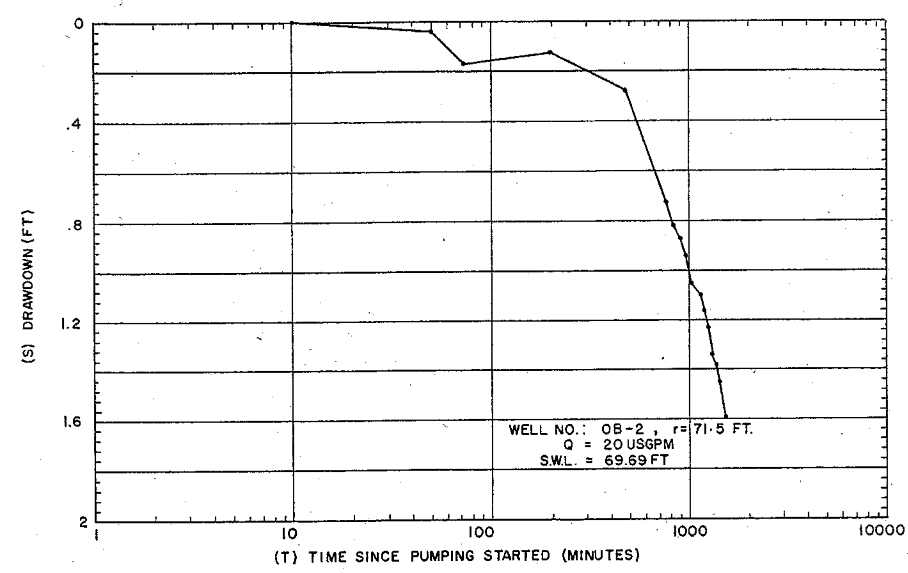

FIG. 9. Time-Drawdown at OB-2, Q = 20 GPM

m) for OB-1 and OB-2, respectively. Water levels in observation well OB-3 actually rose by 0.44 ft (0.13 m) during the 20-gpm (1.26-L/s) test, probably reflecting an antecedent recharge trend or perhaps gas pressure. Gas pressures equivalent to 1.92 ft (58.54 cm), 1.53 ft (46.67 m), and 0.48 ft (14.64 cm) of water head were measured at OB-1, OB-2, and OB-3. Standing water was present on the landfill surface near OB-3 for the duration of the test. It is possible that a recharge trend was also present in the other measured wells and resulted in reduced drawdown measurements. For the 12-gpm (0.76-L/s) test, the drawdown was 3.07 ft (0.94 m) and 1.23 ft (0.38 m) in the two closest observation wells, OB-1 and OB-2, respectively, and the water level at OB-3 continued to rise by an additional 1.16 ft (0.35 m). If the pretest static water levels are considered, the drawdown for the 12-gpm (0.76-L/s) test would be 6.11 ft (1.86 m) and 2.43 ft (0.74 m) for

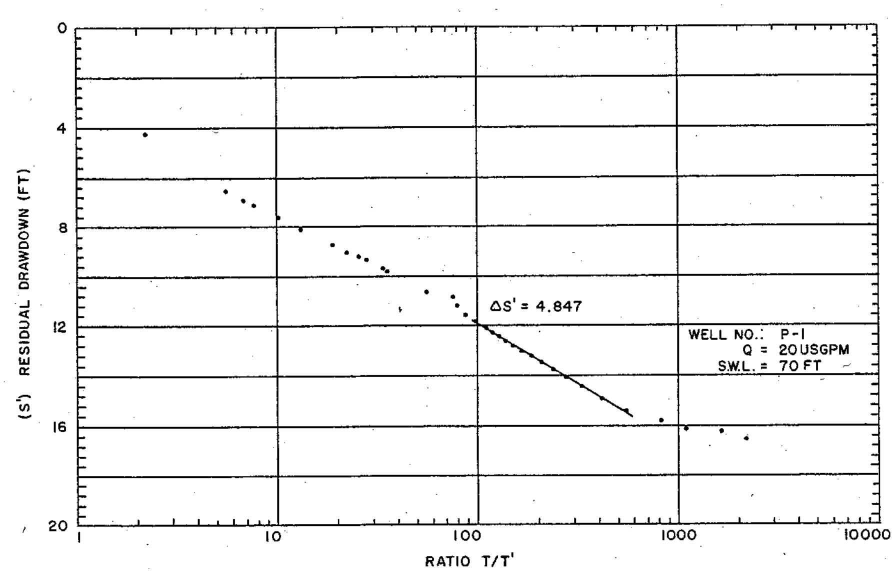

FIG. 10. Recovery Analysis at P-1, Q = 20 GPM

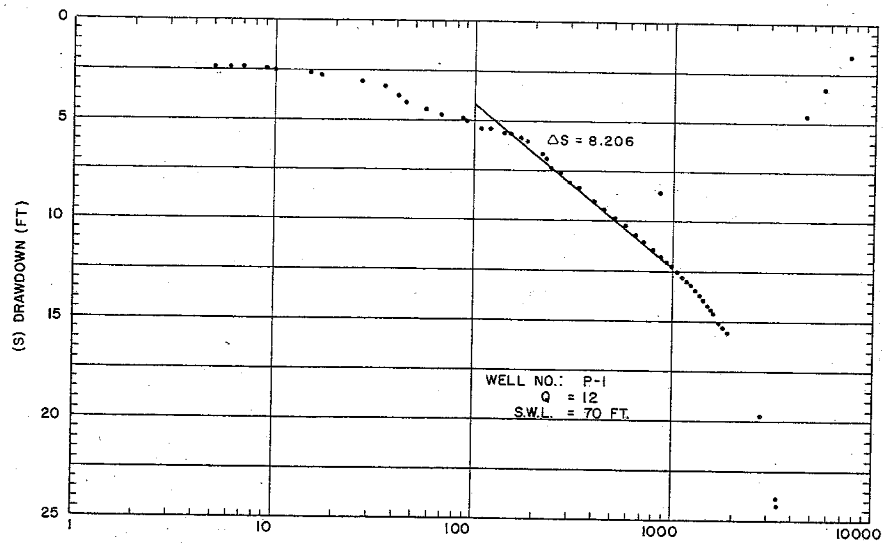

FIG. 11. Pumping Test Analysis, P-1, Q = 12 GPM

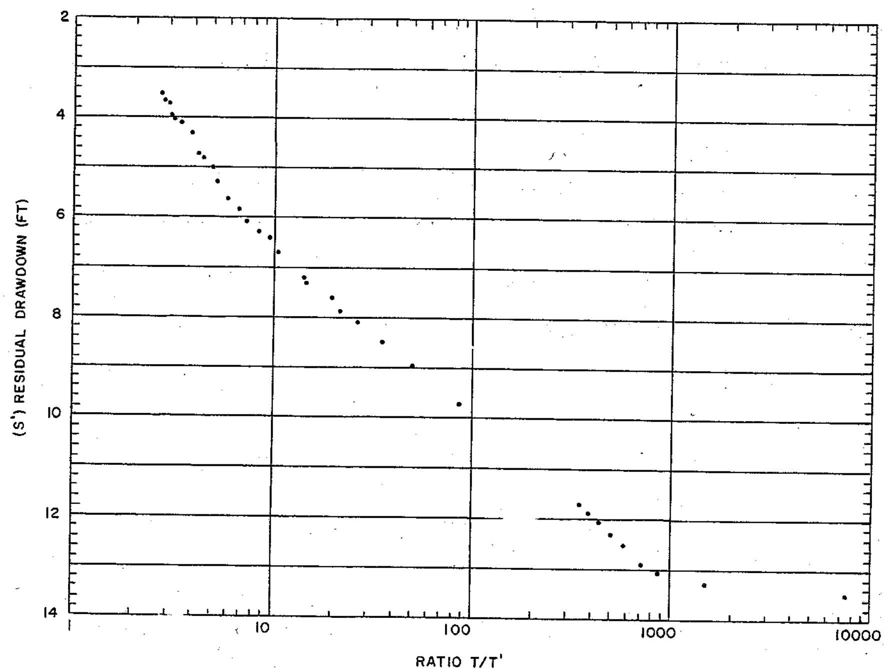

FIG. 12. Recovery Data, P-1, Q = 12 GPM

the OB-1 and OB-2, respectively. The distance-drawdown relationships are plotted in Fig. 13.

Results

All the drawdown plots are typical of the case in which all water comes from storage with no aquifer recharge. The north side of the landfill (a free surface) has influenced the shape of the curves. Even though the saturated refuse and the site geometry contravene many assumptions inherent to conventional hydrogeologic analysis, transmissivity of the refuse was computed to range from about 200 to 1,000 gpd/ft (2.5–12.4 m^2/day), with an average value of about 600 gpd/ft. These values were calculated from time-

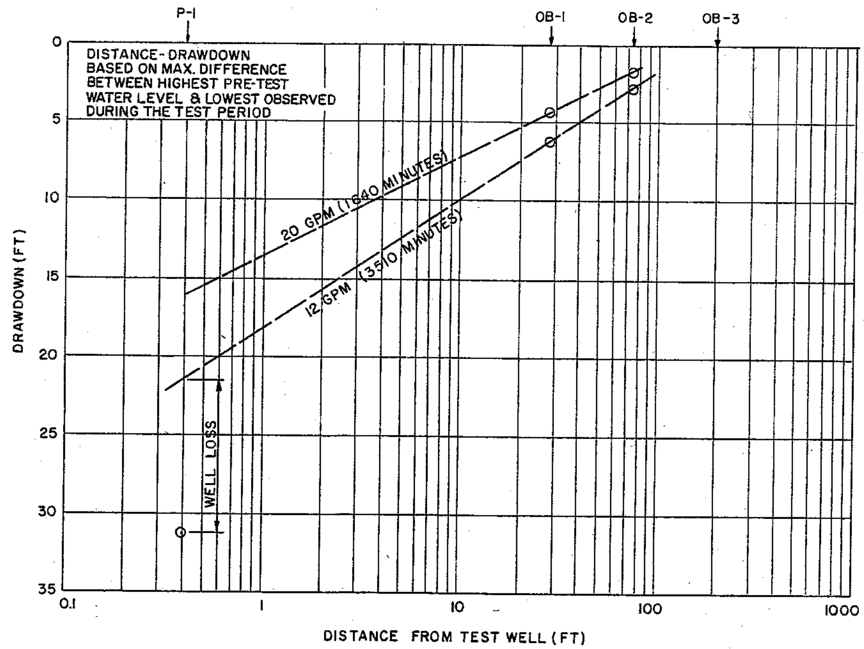

FIG. 13. Constant Rate Pumping Tests: Distance-Drawdown Relationship

drawdown curves and recovery curves for P-1, OB-1, and OB-2. Variations in values relate to the method of analysis and may also be due to heterogeneities within the refuse. If the average saturated thickness is considered to be 30 ft (9.15 m), the corresponding hydraulic conductivity is approximately 20 gpd/sq ft (9.4 $\times$ 10^{-4} cm/s). The transmissivity can also be estimated from the distance drawdown plot of Fig. 13 using the relationship (Driscoll 1986)

$$T = \frac{528Q}{S} \qquad (3)$$

where S = drawdown in feet per 10-fold increase in distance on the log scale.

The calculated transmissivities from Fig. 13 are 633 gpd/ft (7.86 m^2/day) and 1,576 gpd/ft (19.6 m^2/day) for 12 gpm (0.76 L/s) and 20 gpm (1.26 L/s), respectively. Considering an average saturated thickness of 30 ft (9.15 m), the calculated conductivities are 10^{-3} cm/s and 2.46 $\times$ 10^{-3} cm/s for 12 gpm (0.76 L/s) and 20 gpm (1.26 L/s), respectively.

The use of a thick gravel pack in the test well to reduce well losses does not appear to have been effective, as demonstrated by the apparent well loss (Fig. 13). It is possible that overpumping during the step test may have caused partial plugging of the well screen, which contributed to loss of efficiency.

The storage coefficient values, calculated from the drawdown plots for observation wells OB-1 and OB-2, averaged about 0.05. These values are supported by the volume balance method described by Nwankwor et al. (1984). In this method the specific yield is determined from the ratio of the cumulative volume of water pumped to the volume of the water table drawdown cone for short-term pumping. The long-term specific yield of refuse has been assumed to be as high as 0.10. Gravity drainage is believed to be far from complete during relatively short-term pumping tests. The specific yields computed from this pumping test are, therefore, probably less than the long-term specific yield of the refuse. In addition, the specific yield computed from this test largely represents only the dewatered portion of the refuse within the cone of depression and, therefore, may not be representative of the refuse as a whole.

Based on the aforementioned values, unconfined conditions are present within the landfill. It is interesting to note that in normal aquifers under water-table conditions, the water-table surface is assumed to be at atmospheric pressure. Atmospheric pressure is normally not taken into account when assessing the energy available to act as a driving force for unconfined groundwater flow, because it is assumed to be more or less constant at a site. As previously stated, however, gas pressures at the observation wells were between 0.21 and 0.83 psi (1.45 and 5.72 kPa) during the test. These values indicate that between about 0.5 and 2 ft (0.15 and 0.61 m) of additional head is created by the gas within the landfill, which had only daily cover. This may account for the lower than expected specific yield calculated from the pumping test data; i.e., a gas-driven semiconfined condition exists. It can be inferred that gas pressures represent an additional driving force for leachate flow in landfills.

Using curve-matching techniques, delayed yields from storage effects were identified in all the wells. Initially, the refuse was assumed to be relatively homogenous and isotropic. Drilling logs, however, revealed the upper unsaturated portion to consist mainly of paper, with alternating layers of silt and sand placed as daily cover. Drilling progress was slow, especially through the upper material, because the refuse attenuated the energy of the bit. According to the driller, it was like drilling through a rubber ball. With increasing depth, the refuse was noticeably more decomposed and compact. In addition, the incidence of encountering large, hard objects, such as refrigerators, engine blocks, etc., increased with depth. This was presumably due to the relative absence of recycling efforts in the early years of operation.

A conceptual model of this landfill, therefore, is one in which the refuse is highly heterogeneous, anistropic, and very porous. Due to the compaction and decomposition of the refuse with increasing depth, there is an accompanying decrease in hydraulic conductivity—as would be expected from laboratory evidence (see Table 2).

Economic Feasibility

The economic feasibility of pumping leachate from a landfill depends on the scope of other remedial measures and is therefore site specific. Consider, for example, a landfill in northern New Jersey with a surface area of 150 acres (60.7 ha), 12,000 ft (3,658.54 m) perimeter, and a depth of 35 ft to a low permeability stratum around the perimeter. The cost of installing a perimeter leachate collection system and a cutoff wall would be about $12,000,000. The regulations also require the installation of an "impervious" cap that may cost about $130,000/acre ($347,100/ha). Thus, the initial capital cost is $210,000/acre ($518,910/ha). At an interest rate of 9% over a 30-yr period, the annualized premium capital cost is $20,441/acre ($50,509.7/ha). With an average cover maintenance cost of $1,000/

acre/yr (\$2,471/ha/yr), the annualized cost is \$21,441/acre/yr (\$52,980/ha/yr). It is assumed that the cost of maintaining the cutoff wall and the leachate collection pipe is negligible.

Considering a pumping system with wells spaced at 200 ft (61 m) (one well per acre) and a 120-ft (36.58-m) well depth, the capital cost of installation is about \$36,000/well. An ordinary cover is needed at a cost of about \$40,000/acre (\$98,840/ha). Thus, the total initial capital cost is \$76,000/acre (\$187,796/ha), and the annualized premium capital cost is \$7,400/acre/yr (\$18,285.4/ha/yr). The average maintenance cost is estimated to be about \$8,000/yr/well, power cost at about \$430/well/yr [130 ft (39.6 m) total head, 50% efficiency, \$0.20/kWh], and \$500/acre/yr (\$1,235.5/ha/yr) for maintaining the cap. The total annualized cost is \$16,330/acre/yr (\$40,351/ha/yr). Over a 30-year period the savings with a well system would be \$153,330/acre (\$378,878.4/ha).

In this analysis it is assumed the "impervious" cap would not offer a significant advantage over an ordinary cap in terms of leachate generation because of the large total and differential settlement of refuse, and subsequent damage to the cap (Oweis 1988). It should also be noted that with a perimeter leachate collection system, a cutoff wall, and an ineffective cap, the leachate mound within the landfill would continue to exist. The pumping system, however, would lower the mound and control the leachate level as leachate is generated, thus providing an environmentally superior option.

Analyses conducted in the manner described are sensitive to many variables. If, for example, a low-permeability soil is available to shorten the cutoff wall and an expensive cap is not mandated by regulations, then pumping is unlikely to be economically feasible, unless the same wells are used for gas extraction. Furthermore, experience with pumping leachate from within landfills is limited. The writers have not located any published information. Actual maintenance costs for wells in refuse are not available. The costs used are based on judgment and experience with the test presented herein.

Conclusions

The degree of compaction, age and degree of decomposition, gas content and temperature all influence the hydraulic characteristics of refuse. These variables need to be assessed on a case-by-case basis.

It appears that a reasonable estimate of hydraulic conductivity can be made based on published data coupled with assessment of refuse unit weight and field measurements of leachate buildup. It is also clear that leachate could be pumped from refuse using available technology. The hydraulic conductivity determined from the pumping test is about 10^{-3} cm/s. Leachate pumping may offer an attractive cost-effective alternative for leachate management when compared to cutoff walls, toe drains, etc. While municipal refuse has a substantially different composition than typical soils, the laws governing water flow in soils appear to be applicable to refuse on a macroscale basis.

Appendix. References

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