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# CALCULATION SUMMARY SHEET



PROJECT NUMBER: 143418

PROJECT NAME: USEN – Trench 13 Design

DATE: December 5, 2014

CALCULATION NUMBER: C.04 Revision: 1 (March 11, 2016)

**CALCULATION TITLE: LCRS Flow Capacity, Sump Restriction, and Pump Sizing**

**DESCRIPTION OF CALCULATION:**

Calculation to determine LCRS Flow Capacity, Sump Restrictions, and Pump Sizing

**REFERENCES USED:**

1. 300 mil Geocomposite Specifications (Typical)
2. Designing with Geosynthetics, Koerner. 1998
3. GSE Drainage Design Manual (2007)
4. Evaluation of an Alternative Method of Leachate Collection System Design, Hettiaratchi et al, 2009.
5. Physical Properties of Plastic Pipe Used in Reclamation Toe Drains, Bureau of Reclamation. 2009.

**REVIEW COMMENTS:**

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|---|---|---|---|
| CALCULATION MADE BY: | CWK | DATE: | 12/05/14 |
| CALCULATION CHECKED BY: | CAB | DATE: | 02/06/15 |
| CALCULATION REVISED BY: | PAZ | DATE: | 05/07/15 |
| CALCULATION REVIEWED BY: | CAB | DATE: | 05/14/15 |

## PURPOSE OF CALCULATION

Determine the flow capacity of the Leachate Collections and Recovery System (LCRS) in each phase of Trench 13 once it has passed through the Operations Layer. The system must be capable of adequately transmitting leachate through the geocomposite drainage layer and into the LCRS sump, preventing head on the LCRS Floor from exceeding one foot at any point outside the sump. The LCRS sump entry and pump should of sufficient capacity to handle the incoming leachate and remove it from the system without backups onto the LCRS Floor exceeding one foot. The pump in the LCRS Sump must be capable of preventing liquid head buildup in the LCRS sump greater than 1.0 foot.

## METHOD

Task 1: The geocomposite drainage layer of the LCRS is evaluated and designed to manage flow from the design precipitation event.

Task 2: The design of the LCRS sump entry is evaluated to determine if restrictions will result in backups of leachate on the liner.

Task 3: The specifications for a pump, capable of recovering the flow from the sumps, are provided.

## INFILTRATION TO THE LCRS

A 2.26-inch (25-yr, 24-hr) design storm occurring immediately following placement of the 30-inch operations layer over the entire floor area of Phases A through E[1], and before placement of any additional waste, is the primary design factor for the LCRS drainage system. The resulting infiltration into the LCRS drainage layer is based on unsaturated conditions and results in the following flows as determined in **Calculation C.03** of the Trench 13 Engineering Design Report. Results of **Calculation C.03** are presented in the table below.

| Phase | Phase Floor Area ($ft^2$) | Phase Sidewall Area ($ft^2$) | Phase Total Area ($ft^2$) | Infiltration into the LCRS ($gal/min/ft^2$) | Infiltration into the LCRS gal/min/phase |
|---|---|---|---|---|---|
| A & E | 374,012 | 56,358 | 430,370 | 5.34E-05 | 23.0 |
| B, C, & D | 374,012 | 25,274 | 399,286 | 4.17E-05 | 16.7 |

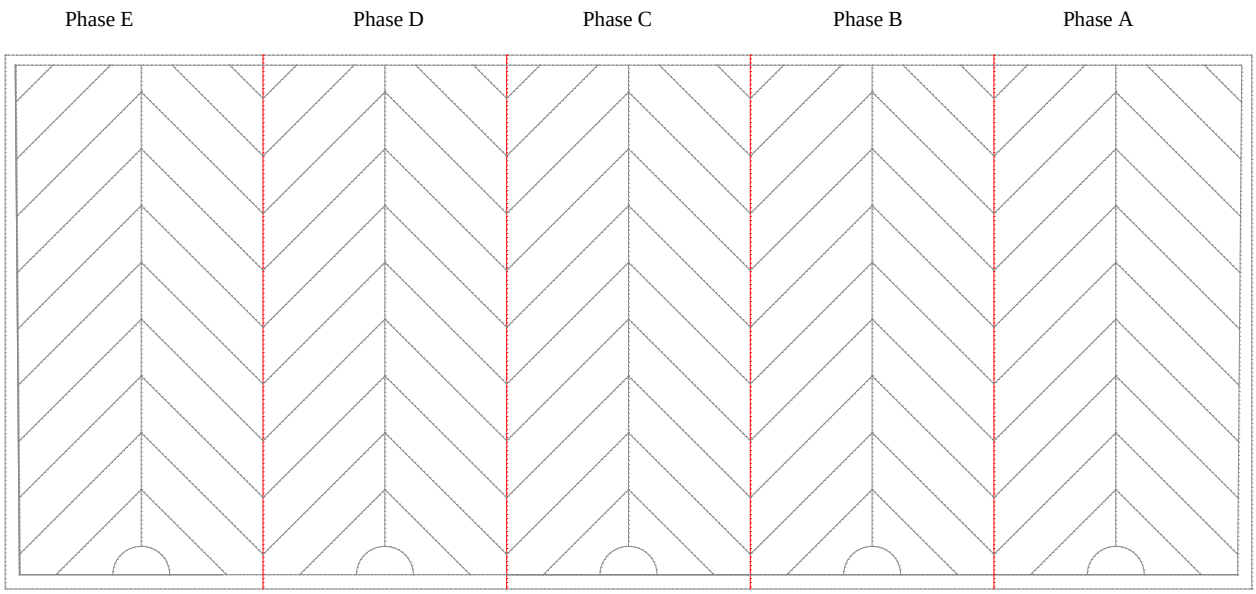
(Calculation C.03 - LCRS Infiltration Rates Calculation)

[1] Considering the entire floor area (including all sidewall slopes) for each of the five phases of Trench 13 development is a conservative assumption as it is likely that the combination of trench phase construction and waste placement operations will not result in the full floor area of any of the phases being covered with only the 30 inch select waste layer at any given time. More likely will be situations where only a fraction of the floor areas is in this condition, either because the full floor area has not been constructed or because additional waste material will have been placed above the select waste over portions of the floor area.

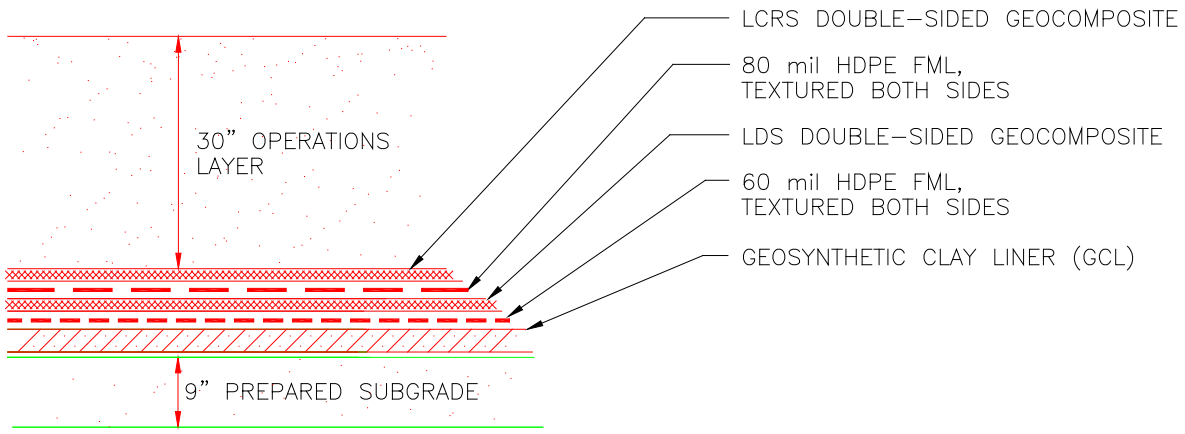
## Task 1: Flow Capacity Through the LCRS Drainage Layer

A single, continuous layer of double-sided geocomposite (300-mil, **References 1a, 1b, and 1c**) is used as the LCRS drainage layer to transmit leachate to the sump for collection and removal. The trench floor layout is shown in **Figure 1**. The components of the Trench 13 floor are shown in **Figure 2**. Flow within in the LCRS is controlled by the minimum bottom slope of the landfill trench floor slope and the geometry of the LCRS sump entry.

### Figure 1 Trench 13 Phase Layout




### Figure 2 Typical Trench 13 Floor Components



## Unit Flow and Hydraulic Conductivity Through the LCRS

Geocomposites considered have a material transmissivity of 4.35 gal/min-ft$^2$ (see **References 1a, 1b, and 1c** for typical double-sided geocomposites with – 8 oz/yd$^2$ geotextile and netting at 300 mil). The relationship between transmissivity and flow is determined using d'Arcy's Law.

D'Arcy Flow = $q_{ult}$ =KiA

Where K = hydraulic conductivity
$i$ = gradient
A = area

Transmissivity = T = $b$*K where $b$ is the thickness of the geocomposite. Rearranging the transmissivity equation:

$$K = \frac{T}{b}$$

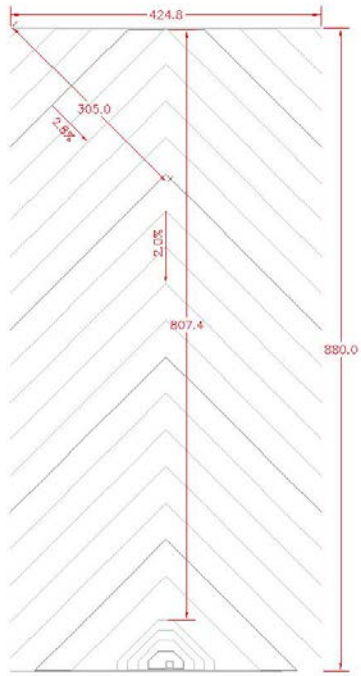
For a (1 ft) unit width of geocomposite:

A = 1 x $b$, therefore A = $b$ ($b$ in feet)

$$q_{ult} = \frac{T}{b} * i * b = T * i \quad \text{or} \quad q_{ult} = T*i$$

Leachate flow lines (in the geocomposite member) from each phase intersect the phase centerline at 45°, resulting in a normal slope of 2.8 percent toward the centerline. Flow along the centerline, toward the Phase sump is at 2.0 percent. The maximum in-plane (i.e., in the geocomposite) travel of any phase of Trench 13 is approximately 305 feet as illustrated in **Figure 3**. Note – a leachate collector pipe runs along the centerline to the sump, so only flow in the LCRS, directly under the collector pipe, will flow at 2.0 percent. Thus evaluations of flow in the LCRS are based on a gradient of 2.8 percent.

**Figure 3 Phase Floor Dimensions and Slopes**




Substituting the specified material transmissivity (4.35 gal/min) per foot of geocomposite width and the minimum bottom gradient of Trench 13 of 2.8 percent.

$q_{ult}$ = 4.35 gal/min * 0.028 ft/ft = 0.12 gal/min through 1 foot of unit width of geocomposite.

Effective flow for the geosynthetic is calculated by considering several factors that reduce the ultimate flow capacity (rate) of the geocomposite, such as elastic deformation ($FS_{IN}$), creep ($FS_{CR}$), chemical clogging ($FS_{CC}$), and biological clogging (FSBC) factors. The following, from Koerner 2012, (**Reference 2**) is used for that calculation of the reduced flow capacity. Because the geonet transmissivity test data from the manufacturer is for short duration testing, the effective flow equation using four reduction factors is applied. Koerner recommends the use of average reduction values in the absence of additional information regarding geocomposite behavior.

$$q_{eff} = \frac{q_{ult}}{(FS_{IN} \times FS_{CR} \times FS_{CC} \times FS_{BC})}$$

where:

$q_{allow}$ = allowable flow rate under long term design conditions
$q_{reported}$ = manufacturer short term test flow rate
$RF_{IN}$ = reduction factor to flow caused by intrusion of adjacent geosynthetics into geonet
$RF_{CR}$ = reduction factor to flow caused by geonet elastic deformation (creep)
$RF_{CC}$ = reduction factor to flow caused by chemical precipitation clogging

$RF_{BC}$ = reduction factor to flow caused by biological clogging

The creep reduction factor for long term application at the final design loading of about 15,000 psf was selected as 1.19 based on literature test values (**Reference 3**: GSE, 2007), so this value was used rather than the average range.

- $FS_{IN}$ = 1.5 to 2.0, use 1.75
- $FS_{CR}$ = 1.19
- $FS_{CC}$ = 1.5 to 2.0, use 1.75
- $FS_{BC}$ = 1.5 to 2.0, use 1.75

The table below shows the unit flow capacity for the LCRS drainage layer in the floor of Trench 13, based on the applicable transmissivity, hydraulic gradient, and reduction factors (RF).

| Flow Element | Θ | Θ | *i* | $q_{ult}$ | $FS_{IN}$ | $FS_{CR}$ | $FS_{CC}$ | $FS_{BC}$ | $q_{eff}$ | $q_{eff}$ |
|---|---|---|---|---|---|---|---|---|---|---|
| Units | $m^2/s$ | gal/min | ft/ft | gal/min/ft | NA | NA | NA | NA | gal/min/ft | gal/day/ft |
| Floor LCRS Geocomposite – 300 mil with double layer of 8oz/yd geotextile | $9x10^{-4}$ | 4.35 | 0.028 | 0.12 | 1.75 | 1.19 | 1.75 | 1.75 | 0.019 | 27 |

Using the effective flow through the geocomposite to determine hydraulic conductivity yields the following:

$$K = \frac{q_{eff}}{Ai}, \text{ for unit area}$$

Where: A = unit width * b
b = 300 mil (two-sided geocomposite) = 0.300 inch
unit width = 1ft
i = 0.028 ft/ft

Area = 300mil/1000 mil/inch x (1ft/12 inch) x 1ft (unit width) = 0.025 $ft^2$

Hydraulic conductivity = $q_{eff}$ / A / i

$$\text{Hydraulic conductivity} = 0.019\frac{gallons}{\min - ft} \times \frac{0.13368\,ft^3}{1gallon} \times \frac{1}{0.025\,ft^2} \times \frac{1\,ft}{0.028\,ft} = 3.6\,ft/\min$$

## Head Buildup in the LCRS

Subject to the conservative assumption presented previously regarding the area of phase floor that would be covered with only Operations Layer (selected waste) at any given time, the peak leachate head on the primary liner system will occur after the minimum initial thickness of waste (30 inches of select waste) has been placed over the phase floor.

Using the maximum design infiltration of 23 gallons per minute for Phases A and E, or 5.34E-05 gallons per minute per $ft^2$ and the previously calculated effective flow rate of 0.0132 gallons per minute per ft, a length of geocomposite, in the plane of flow and capable of transmitting the infiltrating leachate, can be determined:

$$\text{Length of LCRS Geocomposite} = \frac{q_{eff}}{Infiltration} = \frac{0.019\frac{gal}{\text{min}-ft}}{5.34E-05\,\frac{gal}{\text{min}-ft^2}} = 356 \text{ feet}$$

Based on the calculation, the 300 mil double-sided geocomposite of the LCRS can handle the infiltration from a linear section of 356 feet. Since the longest run in the Trench 13 design is 305 feet, backups of leachate are not expected.

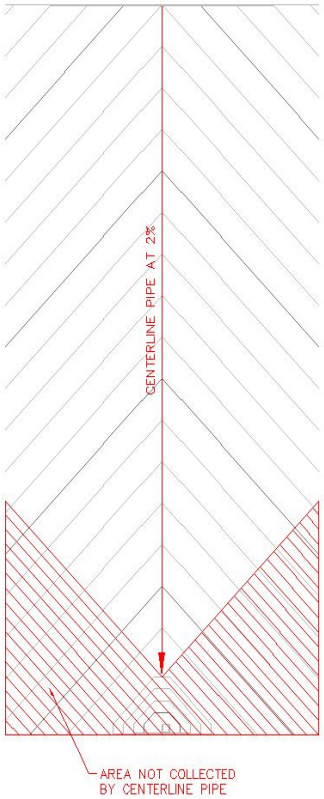
**Task 2: Flow Convergence at the Sump**

**<u>Need for Centerline and Laterals</u>**

For the sump of each Phase to accommodate $q_{eff}$ without leachate 'backing up', the geocomposite cross section entering the LCRS sump perimeter must have a minimum capacity equivalent to the infiltration rate. Without the centerline pipe, that infiltration rate would be 23.0 gallons/minute for Phase A and E and 16.7 gallons/minute for Phases B, C, and D, entering the sump and at an effective flow rate of 0.019 gallons per minute per foot. The needed sump perimeter would be 1210 feet and 879 feet, respectively. Since the sump perimeter is 204 feet, the centerline piping is necessary.

With the centerline pipe, flow convergence at the sump comes from the portion of the LCRS drainage layer that does not flow to the centerline. That portion is represented by shaded area on **Figure 4.** This area flows into the LCRS and to the sump without intersecting the centerline. That drainage area (as highlighted below) is 90,872 $ft^2$ for Phase A and E, and 81,080 $ft^2$ for Phase B, C, and D. As determined below, these values correspond to infiltration rates of 4.84 gal/min/$ft^2$ for Phase A and E and 3.36 gal/min/ $ft^2$ for Phase B, C, and D that do not enter the sump by way of the centerline piping.

**Figure 4 Shaded Area Flows Directly to Sump**




Phases A and E

$$23.0\frac{gallons-ft^2}{\min}\times\frac{1}{430{,}370\,ft^2}\times\frac{90{,}872\,ft^2}{1}=4.84\frac{gallons-ft^2}{\min}$$

Phases B, C, and D

$$16.7\frac{gallons-ft^2}{\min}\times\frac{1}{399{,}286\,ft^2}\times\frac{81{,}080\,ft^2}{1}=3.36\frac{gallons-ft^2}{\min}$$

*Note: Phase B, C, and D shown in figure*

Flow originating in the shaded areas does not intercept the centerline piping and enters the sump through the geocomposite cross section at the perimeter of the sump. The minimum length of LCRS geocomposite needed at the entry to the LCRS sump (i.e., length of geocomposite cross-section needed to prevent flow backup) for Phases A and E is calculated as:

$$Length\ of\ geocomposite\ (Phases\ A\ and\ E)=\frac{Flow\ to\ sump}{Flow\ rate}=\frac{4.84\ \text{gallons/minute}}{0.019\ \text{gal/min/ft}}=255\ \text{feet}$$

The value is slightly less for Phases B, C, and D and is calculated as:

$$Length\ of\ geocomposite\ (Phase\ B, C, and\ D)=\frac{Flow\ to\ sump}{Flow\ rate}=\frac{3.36\ \text{gallons/minute}}{0.019\ \text{gal/min/ft}}=177\ \text{feet}$$

The Trench 13 sumps in all phases have a perimeter of 204 feet, shown in **Figure 5**, at the grade break between 2% and 7.1% slope, therefore, lateral piping is required to drain the shaded areas in Phases A and E.

To improve drainage lateral piping, as displayed in **Figure 6,** is recommended for all Phases, but, based on this calculation, only required for Phase A and E. The lateral

pipes orientated east to west will be 145 feet in length, and pipes oriented north to south will be 218 feet in length to intercept flow and improve drainage to the sump. The pipe material, diameter, and perforations will be consistent with the specifications for the centerline pipe, as explained in the next section of this calculation.

**Figure 5 Typical Sump Plan**

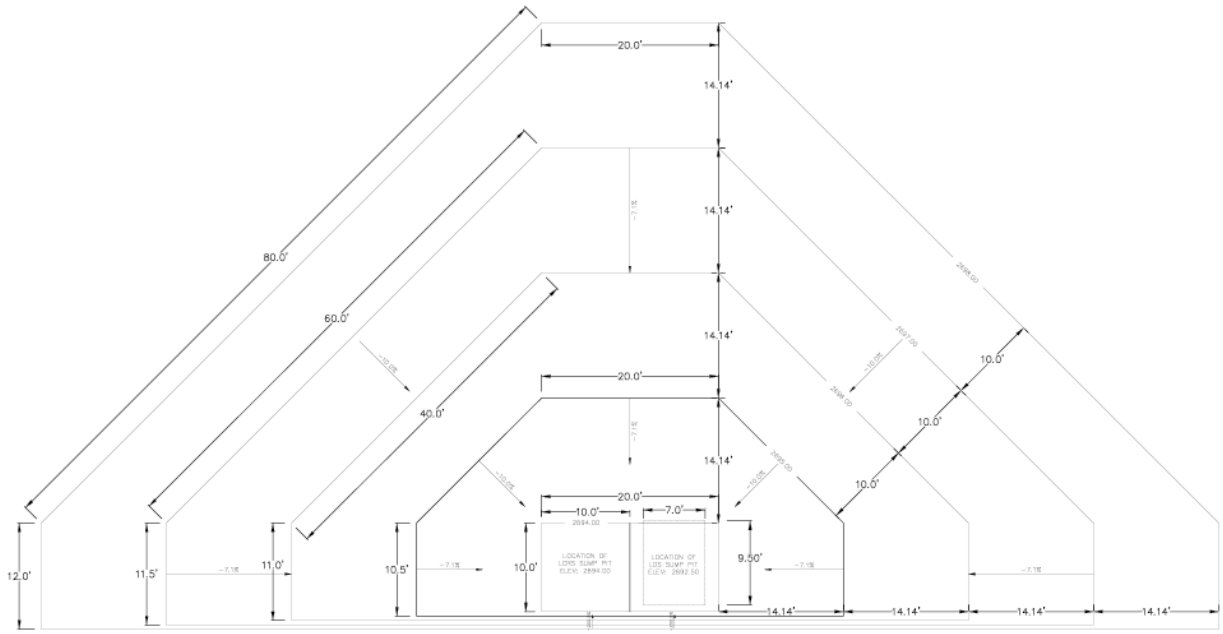
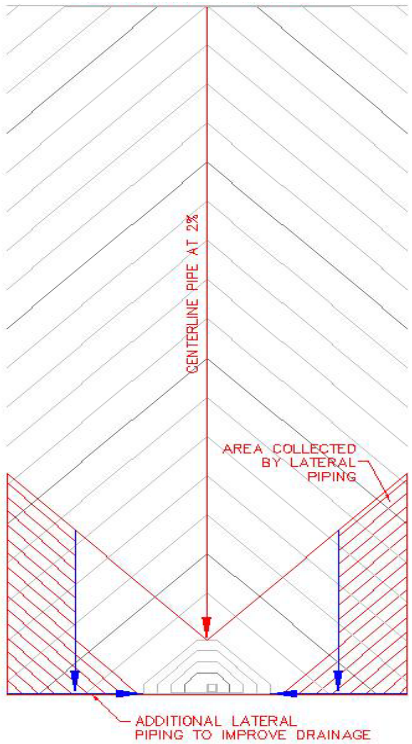



**Figure 6 Recommended Lateral Piping**



## Flow Capacity of Centerline and Lateral Piping

The centerline pipe will be at 2% grade from north to south and must be capable of collecting and carrying flow from a 25-yr 24-hr precipitation event. Manning's equation for flow though a partially full pipe was used for determining flow depth, velocity, and effective discharge to the sump. Bernoulli's principle for orifice flow was used to determine the perforations needed to effectively collect flow from the primary geocomposite liner.

The 25-yr, 24-hr design precipitation event will produce a maximum flow of 23 gallons per minute per phase (**Calculation C.03**). This value will be used to effectively size the centerline pipe and lateral piping for effective transport of leachate from the LCRS to the sump.

Manning's equation for a full-flowing pipe will be used to determine maximum capacity of the centerline pipe and accompanying laterals.

$$Q = \frac{K_n}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} A$$

Where: $Q$ = discharge

$K_n$ = unit conversion factor: 1.49 for English units

$n$ = Manning's roughness coefficient

$R$ = hydraulic radius: $\frac{A}{P_w}$

$A$ = cross-sectional area of pipe

$P_w$ = wetted perimeter inside pipe

$S$ = friction (i.e. energy grade line) slope

The full-flowing capacity of a 4-inch nominal inside diameter pipe:

| Manning's n | | 0.011 | | assumed HDPE | |
|---|---|---|---|---|---|
| Slope | | 0.020 | | ft/ft | |
| Diameter (in) | Area (ft^2) | Wetted Perimeter (ft) | Hydraulic Radius (ft) | Q (ft^3/sec) | Q (gpm) |
| 4 | 0.087 | 1.05 | 0.087 | 0.319 | 143 |

Using Manning's equations, a 4-inch diameter full-flowing pipe will adequately discharge the required capacity of 23.0 gallons per minute. Manning's equation was used again to determine velocity and maximum water depth in the centerline pipe and laterals. The equation was used to evaluate a 4-inch pipe with a maximum flow of 23.0 gallons per minute, slope and roughness coefficient did not change.

| Depth<br><br>in | Depth<br><br>ft | Flow<br>Area<br>ft^2 | Wetted<br>Perimeter<br>ft | Hydraulic<br>Radius<br>ft | Velocity<br><br>ft/sec | Flow<br><br>gpm |
|---|---|---|---|---|---|---|
| Y | Y | A | Pw | Rh | V | Q |
| 1.1 | 0.090 | 0.019 | 0.37 | 0.052 | 2.7 | 23 |

At a maximum flow of 23.0 gallons per minute, a 4-inch diameter pipe will carry leachate at 2.7 feet per second from the LCRS geocomposite to the sump. This value corresponds to 1.1 inches of leachate in the pipe.

The centerline and lateral piping must be able to withstand the normal load imposed by the overlying waste. USEN reports an average placed waste density of 96.3 lbs/ft$^3$ (pcf). For conservatism in calculations, two waste densities were assumed; waste in the upper portion of the cell with a density of a 100 pcf, and waste in the lower portion of the cell that had been subjected to consolidation with a slightly higher density of 115 pcf. With an estimated maximum waste thickness in Trench 13 of 150 feet, the centerline leachate pipe must be able to withstand the combined waste load. For this calculation, it is assumed that the upper 70 feet of waste are at the lower density and the lower 80 feet of waste are at the higher density, producing a load of about 16,200 pound per square foot. Leachate pipe specifications (LCRS Piping Network and Strength **Calculation C.05**) demonstrates that a 4-inch diameter smooth-wall perforated HDPE pipe is adequate for meeting the expected pipe stresses.

The required flow per foot of centerline pipe is calculated from the maximum anticipated discharge and length of pipe. A safety factor for estimating the maximum leachate flow, called the Peak Leachate Factor (PLF), typically ranges from 2 to 4 (**Reference 4:** Hettiarcatchi, et al., 2009). The higher PLF is commonly associated with so called “wet” infiltration events, as would be classified with the 24 hour, 25 year peak storm. Therefore, the maximum PLF = 4 was used to size the leachate collection pipe perforations. So the peak leachate flow becomes:

$$Q = (Infiltration\ Rate)(PLF) = (23\ gpm)(4) = 92\ gpm = 0.205\frac{ft^3}{sec}$$

The total minimum cross-sectional perforated area required to allow the leachate to enter the centerline pipe was calculated using the orifice equation. For this pipe flow, the minimum orifice size can be computed from

$$Q = CA\sqrt{2gH} \qquad \text{and} \qquad A = \frac{Q}{C\sqrt{2gH}}$$

Where: $Q$ = discharge (ft$^3$/sec) = 0.205 ft$^3$/sec

$C$ = coefficient of discharge assumed to be 0.8 for a circular perforation

$A$ = total cross-sectional area of orifice ($ft^2$)

$g$ = gravity (acceleration) = 32.2 $\frac{ft}{sec^2}$

$H$ = head acting on the orifice assumed to be 12.0 inch = 1 ft

Therefore: $A = \frac{0.205 ft3/sec}{0.8\sqrt{2*32.2\frac{ft}{sec2}*1ft}}$

= 0.032 $ft^2$ or 4.6 $in^2$ over the length of the centerline and later pipes.

The centerline pipe, not including laterals, is 807 feet long. The centerline pipe will need minimum perforation area of 0.0057 $in^2$ per foot of pipe.

The Bureau of Reclamation (**Reference 5**: 2009) recommends numerous perforations for low flow applications. The perforation open area must be sized to prevent pipe failure under short –term (operations) and long-term (final waste configuration) conditions. **Calculation C.05** examined pipe strength and percent open area requirements. To allow leachate to enter into the pipe, two lines of perforations at 45 degrees below horizontal are planned. The perforations will be 3/8" diameter with one hole per line per foot, for a total of 2 holes per lineal foot of centerline pipe. This produces an open area of

$$A = 2\pi\frac{d^2}{4} = 2\pi\frac{(0.375\ inches)^2}{4} = 0.22\ in^2$$

This significantly exceeds the minimum perforation requirement of 0.0057 $in^2$, so is acceptable.

**Task 3: Pump Size**

The LCRS pump sizing followed an approach similar to that used for the Trench 12 design (TRC, 1996). For Trench 13, the pump must be capable of lifting the leachate vertically approximately 80 feet (from elevation 2692 to elevation 2768). An additional 10 feet of stickup is needed to account for future final cover. Head losses in the discharge line are expected to be minimal, in the Trench 12 calculation, the designers allowed for 5 feet of loss, as well as minor losses of 20 percent of the subtotaled loss (TRC, 1996). (20% * (88+5), or approximately 19 feet. Total estimated dynamic head (TDH) is therefore:

TDH = 90 ft + 5 ft + 19 ft = 114 feet.

The recommended pump should be capable of providing 114 feet of dynamic head at a rate of 23 gallons per minute. The Peak Leachate Factor was not used in the pump specification because it would lead to an unnecessary oversizing of the pump. The pump could be upsized in the future if necessary.

## SUMMARY

The LCRS calculation examined:

- Flow capacity through the LCRS drainage layer
- Flow convergence at the sump and need for centerline/lateral piping; and
- Pump sizing requirements

The calculation results are summarized below.

Flow Capacity through the LCRS Drainage Layer. A single, continuous layer of double-sided geocomposite (300-mil) is sufficient to transmit leachate to the sump for collection and removal. LCRS drainage material should be 300-mil geonet with two 8 oz/yd geotextiles; the geocomposite transmissivity should be 4.35 gallons per min-ft$^2$.

The maximum in-plane flow length of leachate in the LCRS geocomposite is 305 feet and is less than the allowable in-plane flow length of leachate of 356 ft.

Flow Convergence at the Sump. Centerline piping is required to prevent leachate head buildup on the liner. Flow originating in areas that do not flow to the centerline piping will be conveyed by the geocomposite at the sump perimeter; however, lateral piping in these areas is recommended in this design to further reduce the potential for fluid buildup on the liner.

A 4-inch diameter centerline pipe will need a minimum open perforation with an area of 0.0057 in$^2$ per foot of pipe to allow inflow of the leachate under peak flow conditions. A perforation design of one 3/8-inch diameter hole per foot in two rows and each row located at 45 degrees below horizontal. The pattern will provide 0.22 in$^2$ of perforation and sufficient flow capacity. The pipe strength of such a perforation pattern is examined in a separate calculation (**Calculation C.05**).

Pump Size. Based on the total riser length and anticipated head losses, the LCRS pump should be able to provide 114 feet of dynamic head at a rate of 23 gallons per minute. The safety factor of 4 times the leachate flow was not applied to the pump specification because of the low likelihood of such pumping requirements.

## REFERENCES

**Reference 1: Material Specifications**

**Reference 1a: AGRU 300 mil Geocomposite**

**Reference 1b: GSE 300-mil Geocomposite**

**Reference 1c: SKAPS 300-mil Geocomposite**

**Reference 2: Koerner, Robert 1994. Designing with Geosynthetics**

**Reference 3: GSE Drainage Design Manual (2007)**

**Reference 4: Evaluation of an Alternative Method of Leachate Collection System Design, Hettiaratchi et al, 2009**

**Reference 5: Physical Properties of Plastic Pipe Used in Reclamation Toe Drains, Bureau of Reclamation. 2009**

REFERENCE 1a

# 300 mil Geocomposite

## Geonet Component (1)

| Property | Test Method | Frequency | Minimum Average Roll Value |
|---|---|---|---|
| Thickness, mil (mm) | ASTM D5199 | 50,000 sf | 300 (7.6) |
| Peak Tensile Strength MD, lbs./in. (N/mm) | ASTM D5035/7179 | 50,000 sf | 75 (13.3) |
| Density, g/cm$^3$ | ASTM D792, Method B | 50,000 sf | 0.94 |
| Carbon Black Content (%) | ASTM D4218 | 50,000 sf | 2 - 3 |
| Transmissivity(2), m$^2$/sec. (gal/min/ft) | ASTM D4716 | 500,000 sf | 8 x 10$^{-3}$ (38.6) |

## Geotextile Component(1)

| Property | Test Method | Frequency | Minimum | Average | Roll Value |
|---|---|---|---|---|---|
| Mass per Unit Area, oz./sq. yd. (g/m$^2$) | ASTM D5261 | 100,000 sf | 6.0 (203) | 8.0 (271) | 10.0 (339) |
| Grab Tensile Strength, lbs.(N) | ASTM D4632 | 100,000 sf | 170 (757) | 220 (979) | 270 (1200) |
| Grab Elongation, % | ASTM D4632 | 100,000 sf | 50 | 50 | 50 |
| Trapezoidal Tear, lbs. (N) | ASTM D4533 | 100,000 sf | 65 (289) | 95 (423) | 105 (467) |
| CBR Puncture , lbs (N) | ASTM D6241 | 500,000 sf | 435 (1935) | 600 (2670) | 725 (3230) |
| Puncture, lbs. (N) | ASTM D4833 | 100,000 sf | 90 (398) | 120 (530) | 165 (730) |
| Permittivity(3), sec.$^{-1}$ | ASTM D4491 | 500,000 sf | 1.5 | 1.3 | 1.1 |
| Water Flow, (3) gpm./ft$^2$ (l/min/m$^2$) | ASTM D4491 | 500,000 sf | 110 (4479) | 95 (3895) | 80 (3280) |
| AOS, U.S. Sieve max (mm)(3) | ASTM D4751 | 500,000 sf | 70 (0.212) | 80 (0.180) | 100 (0.150) |

## Geocomposite

| Property | Test Method | Frequency | | Minimum | Average | Roll Value |
|---|---|---|---|---|---|---|
| Ply Adhesion, lbs./in. (g/cm) | ASTM D7005 | 50,000 sf | | 1 (178) | 1 (178) | 1 (178) |
| Transmissivity (2), m$^2$/sec. (gal/min/ft) | ASTM D4716 | 500,000 sf | Double | 9 x 10$^{-4}$ (4.3) | 9 x 10$^{-4}$ (4.3) | 7 x 10$^{-4}$ (3.4) |
| | ASTM D4716 | 500,000 sf | Single | 3 x 10$^{-3}$ (14.5) | 3 x 10$^{-3}$ (14.5) | 2 x 10$^{-3}$ (9.6) |

## Supply Information

| Standard Roll Length(4) | at Fabric Weight | 6-oz | 8-oz | 10-oz |
|---|---|---|---|---|
| Double Sided | | 160 | 150 | 140 |
| Single Sided | | 180 | 180 | 170 |

Notes:
(1) Component properties are prior to lamination
(2) Geonet & Geocomposite. Transmissivity at 21°C, gradient of 0.1, load of 10,000psf, seat time 15 min. between steel plates.
(3) At time of manufacture. Handling may change these properties.
(4) All roll widths are 14.5 feet. All roll lengths and widths have a tolerance of ±1%
(5) UV Resistance after 500 hours for the geotextile componet exhibits 70% strength retained via ASTM D4355

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REFERENCE 1b

PRODUCT DATA SHEET

# GSE FabriNet 300 mil Geocomposite

GSE FabriNet 300 mil geocomposite consists of a 300 mil thick GSE HyperNet geonet heat-laminated on one or both sides with a GSE nonwoven needlepunched geotextile. The geotextile is available in mass per unit area range of 6 oz/yd² to 16 oz/yd². The geocomposite is designed and formulated to perform drainage function under a range of anticipated site loads, gradients and boundary conditions.

**AT THE CORE:**
**A 300 mil thick GSE HyperNet geonet heat-laminated on one or both sides with a nonwoven needlepunched geotextile.**

## Product Specifications

| Tested Property | Test Method | Frequency | Minimum Average Roll Value[1] | | |
|---|---|---|---|---|---|
| **Geocomposite** | | | 6 oz/yd² | 8 oz/yd² | 10 oz/yd² |
| Transmissivity[2], gal/min/ft (m²/sec)<br>Double-Sided Composite<br>Single-Sided Composite | ASTM D 4716 | 1/540,000 ft² | <br>4.3 (9 x $10^{-4}$)<br>14.5 (3 x $10^{-3}$) | <br>4.3 (9 x $10^{-4}$)<br>14.5 (3 x $10^{-3}$) | <br>3.4 (7 x $10^{-4}$)<br>9.6 (2 x $10^{-3}$) |
| Ply Adhesion, lb/in | ASTM D 7005 | 1/50,000 ft² | 1.0 | 1.0 | 1.0 |
| **Geonet Core [1,3] – GSE HyperNet** | | | | | |
| Geonet Core Thickness, mil | ASTM D 5199 | 1/50,000 ft² | 300 | 300 | 300 |
| Transmissivity[2], gal/min/ft (m²/sec) | ASTM D 4716 | | 38.6 (8 x $10^{-3}$) | 38.6 (8 x $10^{-3}$) | 38.6 (8 x $10^{-3}$) |
| Density, g/cm³ | ASTM D 1505 | 1/50,000 ft² | 0.94 | 0.94 | 0.94 |
| Tensile Strength (MD), lb/in | ASTM D 7179 | 1/50,000 ft² | 75 | 75 | 75 |
| Carbon Black Content, % | ASTM D 4218 | 1/50,000 ft² | 2.0 | 2.0 | 2.0 |
| **Geotextile[1,3]** | | | | | |
| Mass per Unit Area, oz/yd² | ASTM D 5261 | 1/90,000 ft² | 6 | 8 | 10 |
| Grab Tensile Strength, lb | ASTM D 4632 | 1/90,000 ft² | 160 | 220 | 260 |
| Grab Elongation | ASTM D 4632 | 1/90,000 ft² | 50% | 50% | 50% |
| CBR Puncture Strength, lb | ASTM D 6241 | 1/540,000 ft² | 435 | 575 | 725 |
| Trapezoidal Tear Strength, lb | ASTM D 4533 | 1/90,000 ft² | 65 | 90 | 100 |
| AOS, US sieve[1] (mm) | ASTM D 4751 | 1/540,000 ft² | 70 (0.212) | 80 (0.180) | 100 (0.150) |
| Permittivity, $sec^{-1}$ | ASTM D 4491 | 1/540,000 ft² | 1.5 | 1.3 | 1.0 |
| Water Flow Rate, gpm/ft² | ASTM D 4491 | 1/540,000 ft² | 110 | 95 | 75 |
| UV Resistance, % retained | ASTM D 4355 (after 500 hours) | per formulation | 70 | 70 | 70 |
| **NOMINAL ROLL DIMENSIONS[4]** | | | | | |
| Roll Width, ft | | | 15 | 15 | 15 |
| Roll Length, ft | Double-Sided Composite<br>Single-Sided Composite | | 180<br>220 | 170<br>220 | 160<br>200 |
| Roll Area, ft² | Double-Sided Composite<br>Single-Sided Composite | | 2,700<br>3,300 | 2,550<br>3,300 | 2,400<br>3,000 |

NOTES:
- [1]All geotextile properties are minimum average roll values except AOS which is maximum average roll value and UV resistance is typical value. Geonet core thickness is nominal value.
- [2]Gradient of 0.1, normal load of 10,000 psf, water at 70˚F between steel plates for 15 minutes. Contact GSE for performance transmissivity value for use in design.
- [3]Component properties prior to lamination.
- [4]Roll widths and lengths have a tolerance of ±1%.

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REFERENCE 1c

# Skaps Industries, Inc.

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Phone (770)564-1857

DRAINAGE PRODUCT DESCRIPTION SHEET

## US Ecology Nevada, Inc.

### TRANSNET 330-2-8

Transnet 330-2-8 is a superior quality drainage media made by extruding two sets of HDPE strands together to form a diamond shaped net. The net is then heat laminated to an 8 ounce non-woven fabric. This three dimensional structure provides excellent planar liquid flow. The Transnet 330-2-8 conforms to the physical property values listed below:

| NET PROPERTY | TEST METHOD | UNITS | MINIMUM AVERAGE ROLL VALUE |
|---|---|---|---|
| Mass Per Unit Area | ASTM D-5261 | $lbs/ft^2$ | 0.300 |
| Thickness | ASTM D-5199 | mils | 330 +/- 30 |
| Density of Polymer | ASTM D-1505 | $g/cm^2$ | 0.94 |
| Carbon Black | ASTM D-1603 | % | 2 |
| Transmissivity (Geocomposite) | ASTM D-4716 | $Gpm/ft^2$ | 4.35 |
| Transmissivity (Geonet) | ASTM D-4716 | $Gpm/ft^2$ | 38.64 |
| Tensile Strength | ASTM D-5035 | lbs/in | 75 |
| Ply Adhesion | GRI GC7 | Lbs/in | 1.0 |

*Transmissivity measured using water at 20 Degrees C with a gradient of 0.02, between 2 steel plates, under a confining pressure of 10,000 psf after fifteen minutes. Values may vary based on dimension of the transmissivity specimen and specific laboratory.

### STYLE GE-180

GE-180 is a superior quality, nonwoven geotextile produced by needlepunching together 100% polypropylene staple fibers in a random network to form a high strength dimensionally stable fabric. The polypropylene fibers are specially formulated to resist ultraviolet light deterioration, and are inert to commonly encountered soil chemicals. The fabric will not mildew, is non-biodegradable, and is resistant to damage from insects and rodents. Polypropylene is stable within a ph range of 2 to 13. GE180 conforms to the physical property values below:

| FABRIC PROPERTY | TEST METHOD | UNITS | MINIMUM AVERAGE ROLL VALUE |
|---|---|---|---|
| Weight | ASTM D-5261 | oz | 8.0 |
| Grab Tensile | ASTM D-4632 | lbs | 220 |
| Grab Elongation | ASTM D-4632 | % | 50 |
| Puncture | ASTM D-4833 | lbs | 120 |
| Water Flow Rate | ATMD D-4491 | $gpm/ft^2$ | 100 |
| Permittivity* | ASTM D-4491 | $sec^{-1}$ | 1.26 |
| AOS | ASTM D-4751 | US Sieve | 80 (max) |

*At time of manufacturing. Handling may change these properties.

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REFERENCE 2

Fourth Edition

# Designing with Geosynthetics

Robert M. Koerner, Ph.D., P.E.

H. L. Bowman Professor of Civil Engineering,
Drexel University and Director, Geosynthetic
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REFERENCE 2



### 4.1.6 Allowable Flow Rate

As described previously, the very essence of the design-by-function concept is the establishment of an adequate factor of safety. For geonets, where flow rate is the primary function, this takes the following form.

$$\mathrm{FS} = \frac{q_{\mathrm{allow}}}{q_{\mathrm{reqd}}} \tag{4.3}$$

where

FS = factor of safety (to handle unknown loading conditions or uncertainties in the design method, etc.),
$q_{\mathrm{allow}}$ = allowable flow rate as obtained from laboratory testing, and
$q_{\mathrm{reqd}}$ = required flow rate as obtained from design of the actual system.

Alternatively, we could work from transmissivity to obtain the equivalent relationship.

$$\mathrm{FS} = \frac{\theta_{\mathrm{allow}}}{\theta_{\mathrm{reqd}}} \tag{4.4}$$

where $\theta$ is the transmissivity, under definitions as above. As discussed previously, however, it is preferable to design with flow rate rather than with transmissivity because of nonlaminar flow conditions in geonets.

Concerning the allowable flow rate or transmissivity value, which comes from hydraulic testing of the type described in Section 4.1.3, we must assess the realism of the test setup in contrast to the actual field system. If the test setup does not model site-specific conditions adequately, then adjustments to the laboratory value must be made. This is usually the case. Thus the laboratory-generated value is an ultimate value that must be reduced before use in design; that is,

$$q_{\mathrm{allow}} < q_{\mathrm{ult}}$$

One way of doing this is to ascribe reduction factors on each of the items not adequately assessed in the laboratory test. For example,

$$q_{\mathrm{allow}} = q_{\mathrm{ult}}\left[\frac{1}{\mathrm{RF}_{IN} \times \mathrm{RF}_{CR} \times \mathrm{RF}_{CC} \times \mathrm{RF}_{BC}}\right] \tag{4.5}$$

or if all of the reduction factors are considered together.

$$q_{\mathrm{allow}} = q_{\mathrm{ult}}\left[\frac{1}{\Pi\mathrm{RF}}\right] \tag{4.6}$$

where

$q_{\mathrm{ult}}$ = flow rate determined using ASTM D4716 or ISO/DIS 12958 for short-term tests between solid platens using water as the transported liquid under laboratory test temperatures,

REFERENCE 2



$q_{allow}$ = allowable flow rate to be used in Eq. (4.3) for final design purposes,
$RF_{IN}$ = reduction factor for elastic deformation, or intrusion, of the adjacent geosynthetics into the geonet's core space,
$RF_{CR}$ = reduction factor for creep deformation of the geonet and/or adjacent geosynthetics into the geonet's core space,
$RF_{CC}$ = reduction factor for chemical clogging and/or precipitation of chemicals in the geonet's core space,
$RF_{BC}$ = reduction factor for biological clogging in the geonet's core space, and
$\Pi RF$ = product of all reduction factors for the site-specific conditions.

Some guidelines for the various reduction factors to be used in different situations are given in Table 4.2. Please note that some of these values are based on relatively sparse information. Other reduction factors, such as installation damage, temperature effects, and liquid turbidity, could also be included. If needed, they can be included on a site-specific basis. On the other hand, if the actual laboratory test procedure has included the particular item, it would appear in the above formulation as a value of unity. Examples 4.2 and 4.3 illustrate the use of geonets and serve to point out that high reduction factors are warranted in critical situations.

**Example 4.2**

What is the allowable geonet flow rate to be used in the design of a capillary break beneath a roadway to prevent frost heave? Assume that laboratory testing was done at the proper design load and hydraulic gradient and that this testing yielded a short-term between-rigid-plates value of $2.5 \times 10^{-4}$ m²/s.

**Solution:** Since better information is not known, average values from Table 4.2 are used in Eq. (4.5).

TABLE 4.2 RECOMMENDED PRELIMINARY REDUCTION FACTOR VALUES FOR EQ. (4.5) FOR DETERMINING ALLOWABLE FLOW RATE OR TRANSMISSIVITY OF GEONETS

| Application Area | $RF_{IN}$ | $RF_{CR}$* | $RF_{CC}$ | $RF_{BC}$ |
|---|---|---|---|---|
| Sport fields | 1.0 to 1.2 | 1.0 to 1.5 | 1.0 to 1.2 | 1.1 to 1.3 |
| Capillary breaks | 1.1 to 1.3 | 1.0 to 1.2 | 1.1 to 1.5 | 1.1 to 1.3 |
| Roof and plaza decks | 1.2 to 1.4 | 1.0 to 1.2 | 1.0 to 1.2 | 1.1 to 1.3 |
| Retaining walls, seeping rock, and soil slopes | 1.3 to 1.5 | 1.2 to 1.4 | 1.1 to 1.5 | 1.0 to 1.5 |
| Drainage blankets | 1.3 to 1.5 | 1.2 to 1.4 | 1.0 to 1.2 | 1.0 to 1.2 |
| Surface water drains for landfill covers | 1.3 to 1.5 | 1.1 to 1.4 | 1.0 to 1.2 | 1.2 to 1.5 |
| Secondary leachate collection (landfills) | 1.5 to 2.0 | 1.4 to 2.0 | 1.5 to 2.0 | 1.5 to 2.0 |
| Primary leachate collection (landfills) | 1.5 to 2.0 | 1.4 to 2.0 | 1.5 to 2.0 | 1.5 to 2.0 |

*These values are sensitive to the density of the resin used in the geonet's manufacture. The higher the density, the lower the reduction factor. Creep of the covering geotextile(s) is a product-specific issue.

REFERENCE 3

GSE ENVIRONMENTAL™

# DRAINAGE

## DESIGN MANUAL - SECOND EDITION

June 2007


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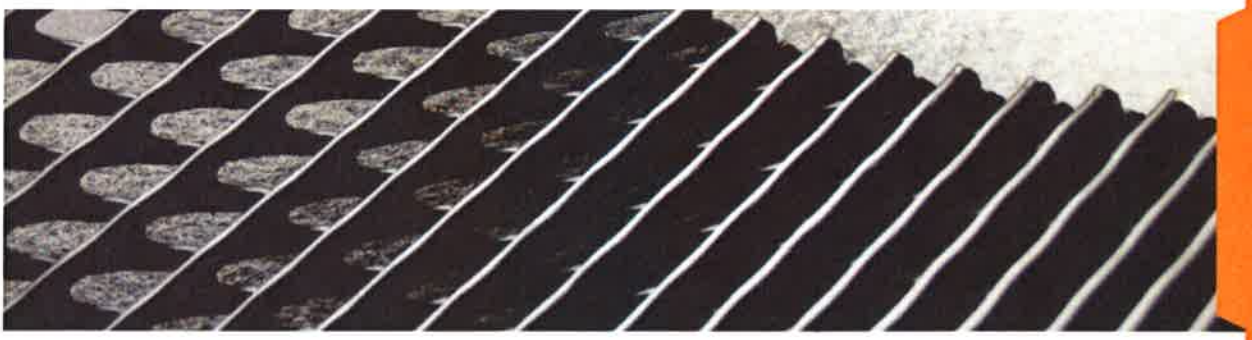

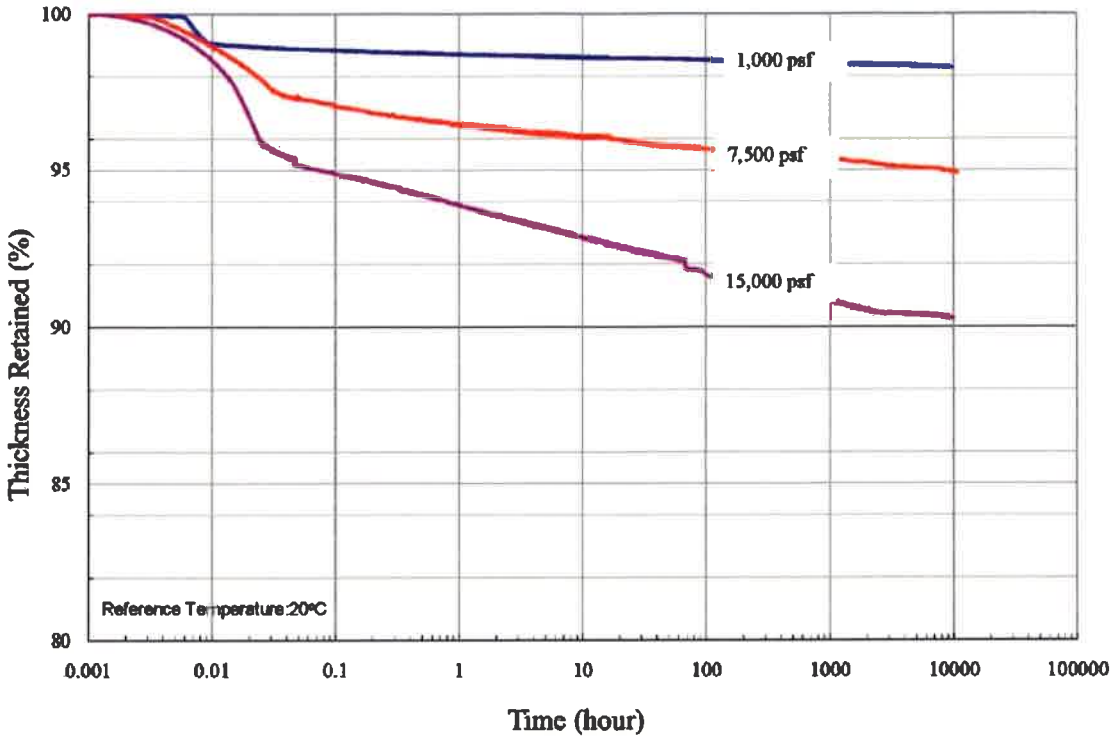


Figure B-3. Creep Curves for a 300 mil GSE HyperNet UF geonet.

Table B-3. Creep Reduction Factors for a 300 mil GSE HyperNet UF geonet from 100 hours to 50 Years.

| Stress (psf) | Creep Reduction Factor |
|---|---|
| 1,000 | 1.00 |
| 5,000 | 1.06 |
| 15,000 | 1.19 |

# Evaluation of an Alternative Method of Leachate Collection System Design

J. P. A. Hettiaratchi[1]; I. E. T. Soh[2]; and C. Hunte[3]

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**Abstract:** The current practice of leachate collection and removal system (LCRS) design for sanitary landfills has limitations because of the uncertainties associated with a key design parameter, leachate production rate. The traditional techniques for determining leachate generation are accurate only to within one order of magnitude. One of the drawbacks is the inability of these techniques to account for short-term fluctuations in leachate generation rate caused by heavy rainfall events or leachate recirculation events. This paper presents a rational method to account for fluctuations in leachate generation by incorporating a correction factor to leachate generation rates determined from traditional techniques. The correction factor known as the peak leachate factor (PLF) is developed from landfill lysimeter and pilot scale test cell studies. It is defined as the ratio between peak and average daily leachate generation rate, and is proposed to be used in conjunction with the output from a leachate generation model such as the hydrologic evaluation of landfill performance model to generate a design leachate generation rate. The potential of applying PLF in the design of a LCRS in full scale landfills is evaluated by studying the scale up possibility of the hydrological behavior of laboratory lysimeters to a field scale landfill test cell. The results demonstrated that the hydrologic behavior of the field test cell is similar to that of the lysimeters. The results indicated that various categories of landfill cells will require the adoption of PLFs depending on the type of waste landfilled and climatic conditions and operational characteristics of the landfills. PLF could be an important design parameter for LCRS in landfills cells, especially those operated with continuous leachate recirculation or located in tropical climates subjected to frequent high intensity rainfall events.

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**CE Database subject headings:** Landfills; Lysimeters; Waste management; Design; Reactors; Biological treatment.

## Introduction

When moisture exceeds the field capacity of the waste matrix it percolates to the bottom of a landfill in the form of leachate. Landfill operating practices such as the type of solid waste accepted by the facility, codisposal of liquid waste, compaction density, and shredding of waste are known to affect leachate production. Acceptance of high moisture waste, such as food waste, as well as liquid waste, decreases the moisture storage capacity of landfilled waste. Further, the presence of a high quantity of dry waste components such as paper and cardboard contributes to the absorbing capacity of waste. The storage and absorbing capacity of waste affect the hydraulic properties of waste such as field capacity and hydraulic conductivity, which govern the moisture movement within landfills, and, thus, affect the quantity of leachate generated in landfills.

It is expected that the higher the field capacity, the higher the breakthrough time of leachate discharge and the lower the leachate production. Beaven and Powrie (1995) defined field capacity quantitatively as the sum of initial moisture content and the absorptive capacity, and qualitatively as the moisture content at which the total absorptive capacity has been utilized and free draining conditions exist. This definition, referred to as theoretical field capacity (TFC), may be satisfactory for homogeneous materials such as soils but with municipal solid waste (MSW) landfills, downward percolation can occur before the field capacity has been exceeded. Additionally, solid waste constituents absorb liquid to different degrees (Campbell 1982).

Hydraulic conductivity is a measure of the ability of a porous medium to conduct liquid (Chen and Chynoweth 1995). A higher hydraulic conductivity of waste will, therefore, result in a higher leachate generation rate. The factors influencing the hydraulic conductivity of soil include particle size, void ratio, composition, fabric, degree of saturation (Lambe 1969), and pore geometry (Hillel 1971). However, it is not possible to characterize solid waste based on these factors due to the heterogeneity of solid waste. Density is related to void ratio, fabric, and pore geometry and, hence, it is important to the hydraulic conductivity of solid waste (Chen and Chynoweth 1995). Tests on household waste have indicated that the saturated hydraulic conductivity is related to density in such a manner that the higher the density, the lower the saturated hydraulic conductivity (Powrie and Beaven 1998). Hydraulic conductivity is also a function of moisture content and increases as the water content increases (Bleiker et al. 1995).

Large quantities of leachate can be generated within a landfill cell depending on the amount of precipitation, moisture content of the waste mass, and final cover design. If not collected and re-

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moved from the base of the landfill, fluid pressures can increase on the bottom liner, and release of contaminants to groundwater can occur posing a hazard to human health and the environment. A leachate collection and removal system (LCRS) is, therefore, a vital component to a properly functioning sanitary landfill. The LCRS consists of a drainage layer, collection pipes, and sumps designed to collect and remove the leachate for treatment or recirculation and to maintain the leachate head above the liner to a specified level, usually 30 cm (USEPA 1999).

The LCRS works in conjunction with a low permeable bottom liner. The landfill designer will manipulate the hydraulic conductivity of the granular material in the drainage layer, the spacing between leachate collection pipes, and the slope of the bottom liner to achieve the LCRS performance objectives (McEnroe 1989). To assist in this process, mathematical equations that can be solved analytically or numerically are used to determine leachate hydrology. One of the key inputs in these equations is the leachate impingement rate or the leachate generation rate. Korfiatis and Demetracopoulos (1986) contended that leachate generation rate is as important as the hydraulic conductivity of the bottom liner, the dominant landfill design parameter. The efforts of landfill researchers over the last two decades have virtually eliminated uncertainties associated with using hydraulic conductivity as a bottom liner design parameter. The same cannot be said about the leachate generation rate.

A variety of water balance methods are available to determine the leachate generation rate at a landfill cell. They rely on a multitude of analytical models to quantify the various hydrologic processes occurring within the landfill cell. The most commonly used tool is a computer model known as the hydrologic evaluation of landfill performance (HELP) model. Although it is highly popular among practicing engineers, the HELP model is not without shortcomings. Nevertheless, because of its ease of operation and lack of a suitable alternative, the HELP model will find users in North America, and worldwide, in years to come.

One of the major limitations of the HELP model is related to the output results. The HELP model simulations result in daily average leachate generation or impingement rates, which may be of limited use to designers of most LCRSs. The leachate generation rate varies with time and is largely affected by the intensity and duration of precipitation events (Sharma and Lewis 1994), which are not captured in the HELP model. To make matters worse, there is no clear consensus among designers on what leachate generation rate should be used as input in LCRS design equations. Depending on the site conditions and regulatory requirements, the designers are known to use average yearly, average monthly, or daily average leachate generation rate determined from the HELP model, or simply use the 100-year design storm to size the LCRS pipes. The Californian landfill regulations require the use of twice the maximum anticipated daily leachate generation rate (Sharma and Lewis 1994). The factor of two, in this case, is supposed to compensate for the uncertainty associated with the calculated leachate generation rate.

With the emergence of leachate recirculating landfills, and, thus, with a greater emphasize on high efficiency leachate collection systems, greater challenges have been imposed on landfill engineers involved in the prediction of leachate generation rates. In this paper, a correction factor developed from lysimeter studies, peak leachate factor (PLF), is introduced. The correction factor PLF is defined as the ratio between peak and average daily leachate discharge rates. The potential to use PLF as a correction factor to modify the HELP model output in determining an accurate leachate generation rate is evaluated by studying the scale up possibility of the hydrological behavior of laboratory lysimeters to the field scale landfill test cell.

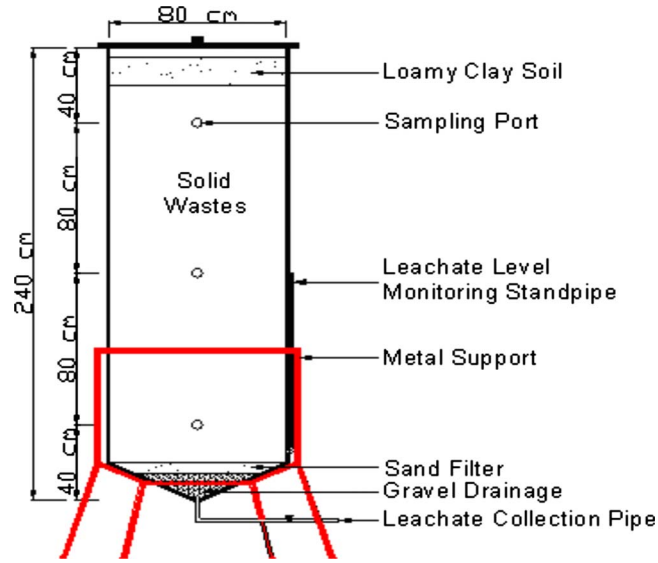


**Fig. 1.** Schematic diagram of laboratory lysimeter arrangement

## Materials and Methods

Two laboratory scale landfill lysimeters and a field scale landfill test cell were used in this study. The lysimeter studies were undertaken to develop time-dependent relationships of liquid infiltration rate versus leachate generation rate. The field scale landfill test cell is used to determine whether results from laboratory scale lysimeters can be scaled up to field operations.

### *Laboratory Landfill Lysimeters*

Two laboratory scale landfill lysimeters (i.e., IL1 and IL2) of identical design located at the Catholic University of Valparaíso Valparaíso, Chile, were used in this research. The lysimeters are of 0.8 m diameter, 2.4 m height, and 1.2 $m^3$ effective volume (see Fig. 1). They have a conical base with an inward slope of 3%. A 12.5 mm PVC leachate collection pipe complete with a valve is connected to the base of the lysimeter. The waste layer is covered with a layer of loamy clay soil. The thickness of the waste and cover soil layers were 1.96 and 0.23 m, respectively, for lysimeter IL1, and 1.88 and 0.27 m, respectively, for lysimeter IL2.

The lysimeters were filled with shredded waste representative of MSW. The composition of the waste placed in the lysimeters is shown in Table 1. Details of the thickness and density of the cover soil and waste layers in the two lysimeters at the beginning of this research project are summarized in Table 2. The initial densities of the soil and waste layers were determined using a neutron probe. The neutron probe was inserted into four different depths −5, 10, 15, and 20 cm of the soil layers to obtain the density at each level. The neutron probe was also inserted into the middle sampling port of the lysimeters to determine the initial density of the waste. The middle sampling port was chosen because the upper port and the lower port were not suitable for density measurement. The measured waste densities for lysimeters IL1 and IL2 were 814 and 851 $kg/m^3$, respectively, on wet weight basis, and 772 and 770 $kg/m^3$, respectively, on dry weight basis.



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**Table 1.** Composition of Waste in Lysimeters [Modified from Espinace Abarzúa and Schiappacasse Dasati (1997)]

| Waste type | Weight (kg) | Percentage by weight (%) |
|---|---|---|
| Food and yard waste | 244.05 | 48.81 |
| Paper | 68.5 | 13.7 |
| Plastic | 49.3 | 9.86 |
| Cardboard | 24.8 | 4.96 |
| Wood | 2.4 | 0.48 |
| Rubber, leather | 12.6 | 2.52 |
| Textile | 21.75 | 4.35 |
| Metal | 14.9 | 2.98 |
| Glass | 13.8 | 2.76 |
| Dust, ash, etc. | 12.6 | 2.52 |
| Other | 35.3 | 7.06 |
| Total | 500 | 100 |

### *Description of Field Test Cell*

The landfill test cell was constructed in June 1999 and was used in this research from June 2001 to Oct. 2001. The test cell is located at the east Calgary landfill, Calgary, Alberta, Canada. The plan and cross-sectional views of the test cell are shown on Fig. 2. The dimensions of the landfill test cell (TC) are 29.5 m (W) × 37 m (L) × 3.75 m (D) and a base area of 7 m × 7 m (City of Calgary 1999). The side slopes are 1:3 at the transverse sections and 1:4 at the longitudinal sections. The cell base is sloped at 3% towards the center to facilitate leachate collection. The base of the test cell is constructed with a composite liner system containing an 80 mil thick high-density polyethylene geomembrane liner and a 900 mm thick compacted clay layer. The test cell was equipped with a leachate collection system with a drainage layer, perforated piping embedded in a trench and a manhole. The drainage layer of leachate collection system consisted of a 300 mm thick layer of 40 mm washed gravel with nonwoven geotextile filter fabric placed under it and geotextile fabric placed over it. The leachate collection pipe trench was constructed with a bedding of sand layer above which a 150 mm diam pipe (both perforated and nonperforated) was placed. The trench was filled with 40 mm screened gravel and the perforated pipe was wrapped with geotextile. The leachate collection pipes were laid at a slope of 0.8%. The perforated pipe served to collect leachate, and the nonperforated pipes served to drain the collected leachate to the leachate collection sump or manhole. The leachate collection sump was constructed of standard concrete manhole barrels with a minimum of 1 m thick compacted clay layer at the base, and completed with a standard manhole cover.

The test cell was filled with solid waste representative of typical residential and commercial waste in the city of Calgary (Perera 2001). The density and initial moisture content of the waste at the beginning of the research project are estimated to be 593 kg/m$^3$ and 0.22 (vol/vol), respectively. Table 3 presents the composition of the waste used in the test cell.

The average thickness of waste was 1 m. The initial final cover of the cell consisted of a 600 mm compacted clay cap, 300 mm uncompacted subsoil and a 200 mm top soil. However, prior to subjecting the test cell to infiltration events, the cover was partly removed, leaving an average thickness of 500 mm of soil material.

**Table 2.** Thickness and Density of Cover Soil and Waste Layers

| | IL1 | IL2 |
|---|---|---|
| Thickness of cover soil layer (cm) | 23 | 27 |
| Average density of the cover soil (kg/m$^3$) | 1,496 | 1,231 |
| Thickness of waste layer (cm) | 196 | 188 |
| Average density of the waste (kg/m$^3$, wet) | 814 | 851 |

### *Infiltration Experiments—Landfill Lysimeters*

The laboratory lysimeters and the test cell were subjected to infiltration experiments representative of the rainfall events in the regions they were installed. Rainfall events were simulated by spraying tap water on top of the lysimeters and the test cell. A

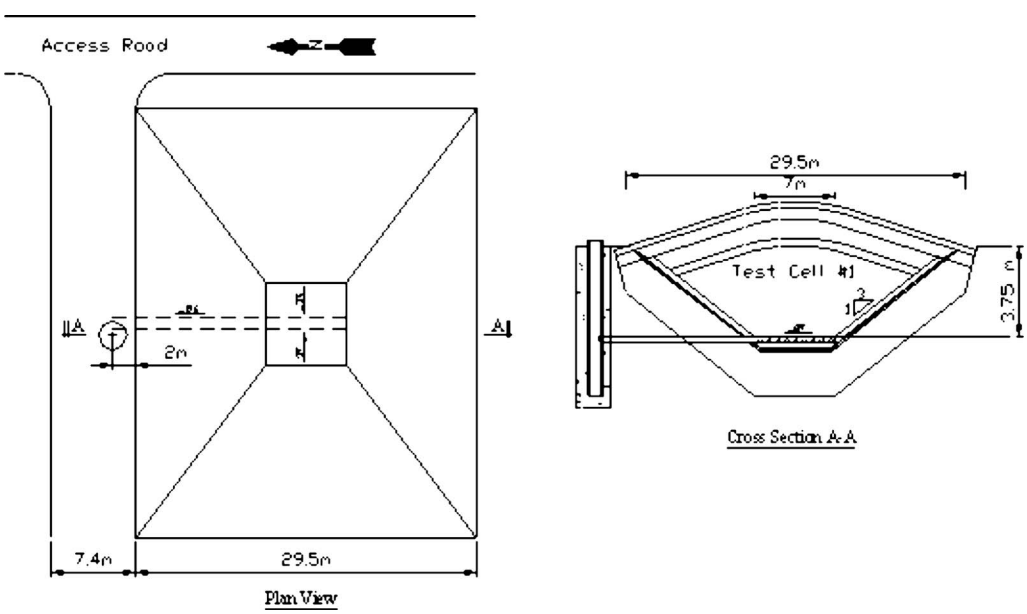


**Fig. 2.** Schematic diagram of the landfill test cell [modified from Perera (2001)]



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**Table 3.** Estimated Waste Composition of the Test Cell [Source Perera (2001)]

| | Multifamily residential waste | | Commercial waste | | Total waste | |
|---|---|---|---|---|---|---|
| Waste types | Weight (kg) | % weight | Weight (kg) | % weight | Weight (kg) | % weight |
| Paper | 343,494 | 54 | 136,491 | 32 | 479,985 | 45 |
| Food waste | 82,693 | 13 | 63,980 | 15 | 146,673 | 14 |
| Plastic | 44,527 | 7 | 34,123 | 8 | 78,650 | 7 |
| Glass | 25,444 | 4 | 12,796 | 3 | 38,240 | 4 |
| Metals | 44,527 | 7 | 8,531 | 2 | 53,058 | 5 |
| Yard waste | n/a | n/a | 110,899 | 26 | 110,899 | 10 |
| Others | 95,415 | 15 | 59,715 | 14 | 155,130 | 15 |
| TOTAL | 636,100 | 100 | 426,535 | 100 | 1,062,635 | 100 |

total of eight simulated infiltration events were performed by subjecting the lysimeters to hydraulic loading rates equivalent to 1-hour 20-year, 5-year, and 2-year storm events expected in Valparaíso, Chile. The rainfall infiltration events conducted using the lysimeters were carried out to study their hydrological behavior with respect to hydraulic loading rates. Table 4 summarizes the information related to the infiltration events conducted using the laboratory lysimeters.

The lysimeters were subjected to different hydraulic loading rates in order to determine the difference in hydrological behavior of the lysimeters in response to differing loading rates. The series of experiments conducted included two infiltration events of the same loading rates (events IL1-R1 and R2, and IL2-R1 and R2), but at different initial moisture contents, on each lysimeter. These experiments were conducted to determine the effect of moisture content on hydrological behavior of the lysimeters. Repeatability of experiments was evaluated by subjecting the two lysimeters, with the same initial moisture content, to the same hydraulic loading rates.

Once water was added to each lysimeter at hydraulic loading rates specified in the experimental design, the leachate production at the bottom of the lysimeter was monitored. The addition of water to the top of the lysimeter continued for 1 h to simulate an infiltration event. Leachate collection, once started, continued every 15 min initially, and then the monitoring period was extended depending on the quantity of leachate produced. A sufficient amount of data was collected to observe the variation of leachate generation rates with respect to a 24-h time series. The plot of leachate generation rate versus time is referred to as a leachate discharge hydrograph (or leachate hydrograph). In addition to generating leachate hydrographs, peak and average leachate production rates in a 24-h period, percentage leachate generation, lag time, and peaking time were also determined. A typical leachate hydrograph is shown in Fig. 3 and resembles typical rainfall-runoff curves observed in surface hydrological studies, with clearly defined lag periods, peaking time, peak flow (or flow amplitude), and flow durations.

### Infiltration Experiments—Field Test Cell

In the case of the field test cell, experiments were conducted by applying water on the cell surface by a combination of spray irrigation and trench infiltration. Spray irrigation was accomplished by pumping stream water through a network of sprinklers. The average hydraulic loading rate applied was 4.37 mm/h and the infiltration period was 2 h. These rates were selected based on the typical precipitation range of $6.1 \pm 8.7$ mm per rainfall event recorded in Calgary, assuming that the precipitation occurred over a 2-h time period. In total, 10 infiltration experiments were conducted. Each experimental infiltration run involved infiltrating a predetermined amount of water at a predetermined loading rate over a time period of 2 h. After each infiltration event, both the volume of leachate generated and leachate generation rate were monitored for a minimum period of approximately 24 h. Table 5 summarizes the hydraulic loading rates applied.

Leachate hydrographs were produced, showing peak leachate generation rate and percentage leachate generation; lag time and peaking time were also determined.

### Leachate Generation Potential

Leachate generation potential (LGP) is defined as the ratio of instant moisture content (IMC) to theoretical field capacity (TFC). The TFC is the ultimate capability of waste to absorb moisture. It

**Table 4.** Details of Infiltration Experiments—Laboratory Lysimeters

| Lysimeter | Event | Event description | Hydraulic loading rate |
|---|---|---|---|
| IL1 | IL1-R1 | Simulate 1-h, 20-year storm event | 240 L/h (or 476 mm/h[a]) |
| | IL1-R2 | Repetition of IL1-R1 | 240 L/h (or 476 mm/h[a]) |
| | IL1-R3 | Simulate 1-h, 5-year storm event with 42% losses due to runoff[b] | 97.8 L/h (or 194 mm/h[a]) |
| | IL1-R4 | 1-h, 2-year storm, with 50% losses due to runoff[b] | 60 L/h (or 119 mm/h[a]) |
| IL2 | IL2-R1 | Simulate 1-h, 5-year storm | 168 L/h (or 338 mm/h[a]) |
| | IL2-R2 | Repetition of IL2-R2 | 168 L/h (or 338 mm/h[a]) |
| | IL2-R3 | Simulate 1-h, 2-year storm | 120 L/h (or 235 mm/hr[a]) |
| | IL2-R4 | Repetition of IL1-R4 | 60 L/h (or 119 mm/h[a]) |

[a]Normalized by the area of lysimeter(s).

[b]Losses due to runoff were accounted for by applying a percentage reduction to the hydraulic loading rates.



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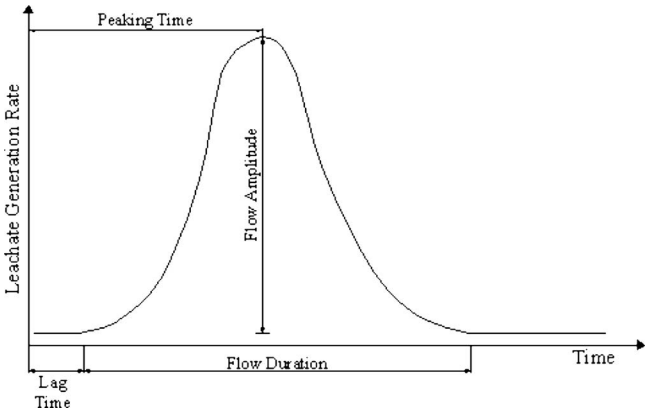


**Fig. 3.** Typical leachate hydrograph

is quasi-constant and does not change for the same waste mass over a short time period. IMC is defined as the moisture content of waste just after an infiltration event was completed. The ratio of IMC to TFC, thus, reflects the potential of leachate generation.

**Table 5.** Details of Rainwater Infiltration Events—Field Test Cell

| Event designation | Hydraulic loading rate |
|---|---|
| TC-R1 | 4.05 $m^3/h$ (or 3.71 $mm/h^a$) |
| TC-R2 | 4.61 $m^3/h$ (or 5.03 $mm/h^a$) |
| TC-R3 | 5.24 $m^3/h$ (or 4.80 $mm/h^a$) |
| TC-R4 | 5.46 $m^3/h$ (or 5.00 $mm/h^a$) |
| TC-R5 | 4.45 $m^3/h$ (or 4.08 $mm/h^a$) |
| TC-R6 | 4.64 $m^3/h$ (or 4.25 $mm/h^a$) |
| TC-R7 | 3.37 $m^3/h$ (or 4.01 $mm/h^a$) |
| TC-R8 | 4.17 $m^3/h$ (or 3.82 $mm/h^a$) |
| TC-R9 | 4.57 $m^3/h$ (or 4.19 $mm/h^a$) |
| TC-R10 | 3.55 $m^3/h$ (or 3.26 $mm/h^a$) |

[a]Normalized by the surface area of the test cell.

Theoretically, when the moisture content of a given waste mass exceeds its TFC, leachate production commences. As such, it can be inferred that a LGP value of greater than unity is required for the appearance of leachate at the bottom of a lysimeter. Leachate generation at a LGP lower than unity reflects a higher degree of short circuiting. The higher the moisture content of the waste mass, the higher the potential it has for leachate generation. As such, it could also be inferred that the higher the LGP value of a given waste mass, the higher will be the potential for leachate generation.

The moisture content of the waste before each infiltration experiment was determined and recorded. By adding the amount of moisture applied during each experiment to the moisture content of waste before each infiltration experiment, the value of IMC of the waste mass was determined.

### *Peak Leachate Factor*

The parameter PLF is determined by dividing the peak leachate generation rate by the average leachate generation rate during an infiltration experiment. The peak leachate generation rate was defined as the maximum leachate generation rate observed during a monitoring period. PLF is an indicator of the variation between peak leachate generation rate and average leachate generation rate and the shape of lysimeter leachate hydrograph. The higher the PLF, the larger will be the variation between peak leachate generation rate and the average leachate generation rate, and, therefore, the sharper the lysimeter leachate hydrograph.

## Results and Discussion

### *Flow Behavior—Laboratory Lysimeters*

The leachate hydrographs produced from the different infiltration experiments involving the lysimeters are presented in Figs. 4 and 5. They follow a typical pattern with defined lag time, flow dura-

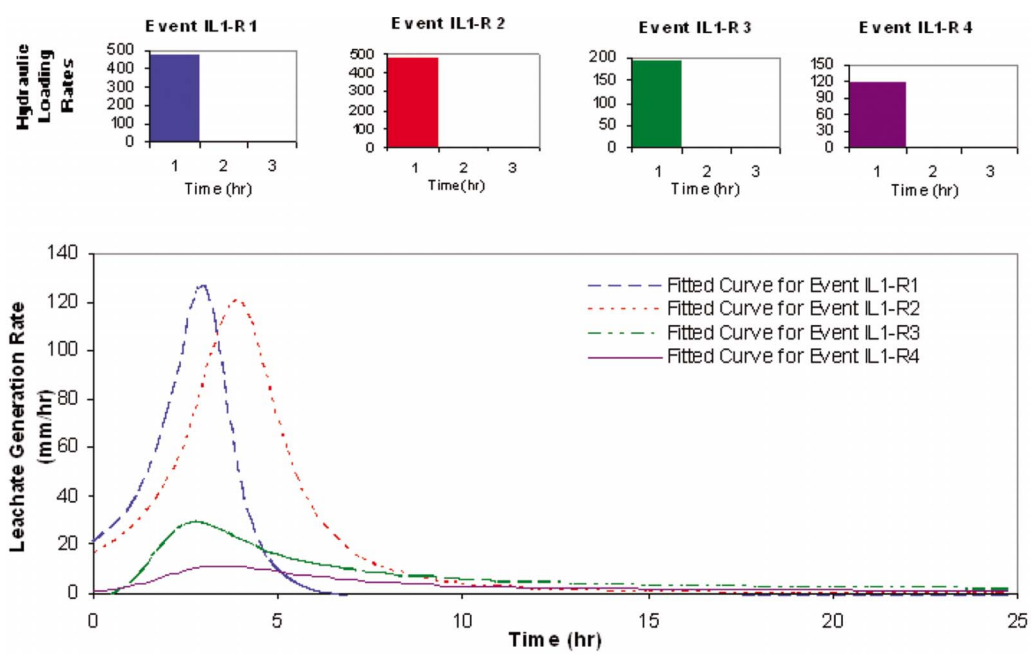


**Fig. 4.** Leachate hydrograph of events IL1-R1–R4



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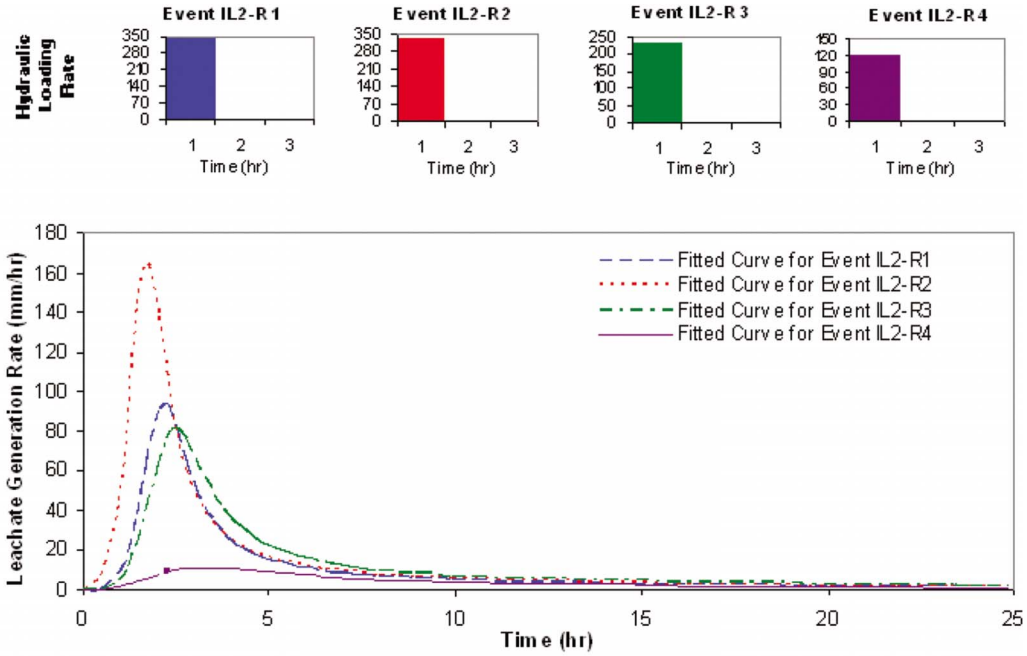


**Fig. 5.** Leachate hydrograph of events IL2-R1–R4

tion, peaking time, and flow amplitude, as in Fig. 3, and resemble typical rainfall-runoff curves observed in surface hydrological studies. The LGP and PLF values determined from each infiltration experiment are presented in Table 6.

As indicated in Table 6, both lysimeters exhibited relatively high LGP values. The LGP of lysimeter IL1 was higher than that of lysimeter IL2, indicating that lysimeter IL1 was wetter than IL2. As such, the lysimeter IL1 represented a wet landfill and IL2 represented a relatively drier landfill. Wet landfills are the ones located in geographical areas where waste moisture contents reach their field capacities rapidly and are subjected to frequent high-intensity infiltration events. Furthermore, bioreactor landfills, or leachate recirculating landfills, fall into this category.

The relationships of PLF and hydraulic loading rates are shown on Figs. 6 and 7, for IL1 and IL2, respectively. Evidently, PLF values are found to be linearly correlated with hydraulic loading rates. It is interesting to note that the range of PLF values is closely correlated with the hydraulic loading rates, irrespective of the lysimeter used. Furthermore, the PLF values of Lysimeters IL1 and IL2 are also found to be a function of LGPs. The dependency of PLF on LGPs is depicted in Figs. 8 and 9.

These results point to the fact that PLF could be used as a potential design parameter in the design of leachate collection and removal systems in wet landfills. In wet landfills and bioreactor landfills, the hydraulic loading rate is the dominant controlling factor of leachate generation and LGP of wastes could exceed the potential of leachate generation easily. The design leachate generation rate for such landfills could be determined by multiplying

**Table 6.** Leachate Generation Potential (LGP) and Peak Leachate Factor (PLF)—Lysimeter Infiltration Experiments

| Lysimeter | Event | LGP | PLF |
|---|---|---|---|
| IL1 | IL1-R1 | 1.491 | 7.87 |
| | IL1-R2 | 1.495 | 7.66 |
| | IL1-R3 | 1.503 | 4.40 |
| | IL1-R4 | 1.503 | 3.43 |
| IL2 | IL2-R1 | 0.832 | 9.64 |
| | IL2-R2 | 0.849 | 12.87 |
| | IL2-R3 | 0.834 | 8.29 |
| | IL2-R4 | 0.826 | 4.05 |

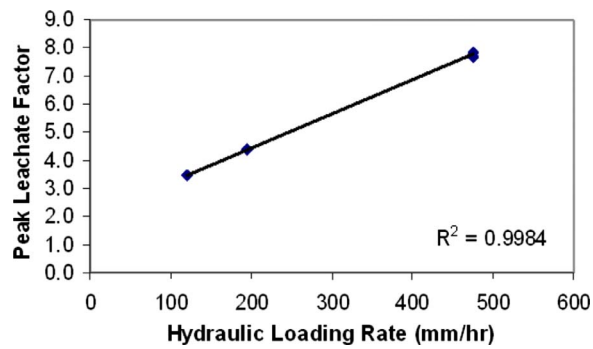


**Fig. 6.** Peak leachate factor (PLF) versus hydraulic loading rate—IL1-R1–R4

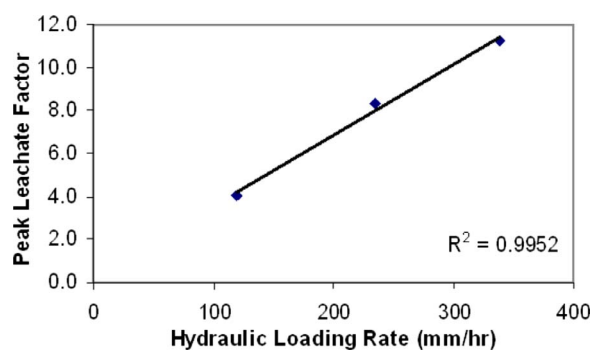


**Fig. 7.** Peak leachate factor (PLF) versus hydraulic loading rate—IL2-R1–R4



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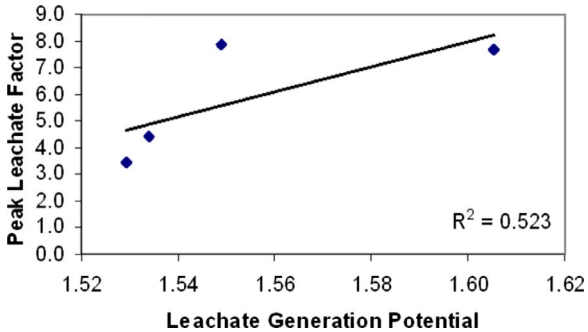


**Fig. 8.** Peak leachate factor (PLF) versus leachate generation potential (LGP)—IL1-R1–R4

the leachate generation rate calculated from a conventional model such as the HELP model. PLF could be considered the safety factor that accounts for fluctuations in leachate generation rate as indicated in a leachate hydrograph. These findings are verified by the results obtained from the field scale test cell studies.

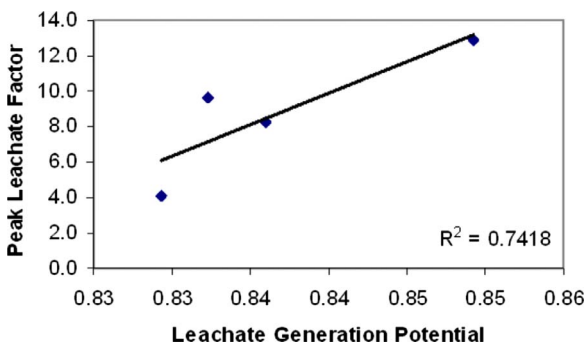


**Fig. 9.** Peak leachate factor (PLF) versus leachate generation potential (LGP)—IL2-R1–R4

### *Flow Behavior—Field Test Cell*

The data collected from the 10 infiltration experiments conducted on the field test cell were used to produce the leachate hydrographs presented in Fig. 10. As depicted in Fig. 10, clear leachate hydrographs with defined peak leachate generation rates were produced when the test cell was subjected to continuous or frequent infiltration events. The first two events, TC-R1 and TC-R2, produced flat leachate hydrographs, with very low peak leachate generation rates. The low peak leachate generation rates were the result of dry initial conditions observed in the final cover soil and possibly, within the cell waste matrix itself. The subsequent infiltration event, TC-R3, produced a well defined leachate hydrograph, similar to the ones observed with lysimeter experiments, with a relatively high peak leachate generation rate.

The subsequent infiltration experiments, undertaken after a short time interval of 26 h, produced a clearly defined leachate hydrograph for event TC-R5. This leachate hydrograph extended for more than 100 h. The following event, TC-R6, also had a discernible leachate hydrograph. The time interval between events TC-R6 and TC-R7 was 146 h, and, consequently, a significant reduction in peak leachate generation rate was observed. The time periods between events TC-R7 and TC-R8, and TC-R8 and TC-R9, were 360 h and 165 h, respectively.

The long time intervals between events have resulted in smaller peak leachate generation rates and flat leachate hydrographs. With a shorter time interval between events, the leachate hydrograph of event TC-R10 exhibited a somewhat discernible leachate hydrograph with a relatively small peak leachate generation rate.

The leachate hydrographs produced from infiltration experiments conducted at the test cell show a clear difference between the hydrologic behavior of a waste cell subjected to frequent and infrequent infiltration events. The appearance of a clearly defined leachate hydrograph with a discernible peak leachate generation rate is contingent upon the supply of water to the waste mass on a continuous basis. A high frequency of infiltration events produced leachate hydrographs with high peak leachate generation rates, indicating the importance of the moisture content of the cell mass in determining the peak leachate generation rate. When the

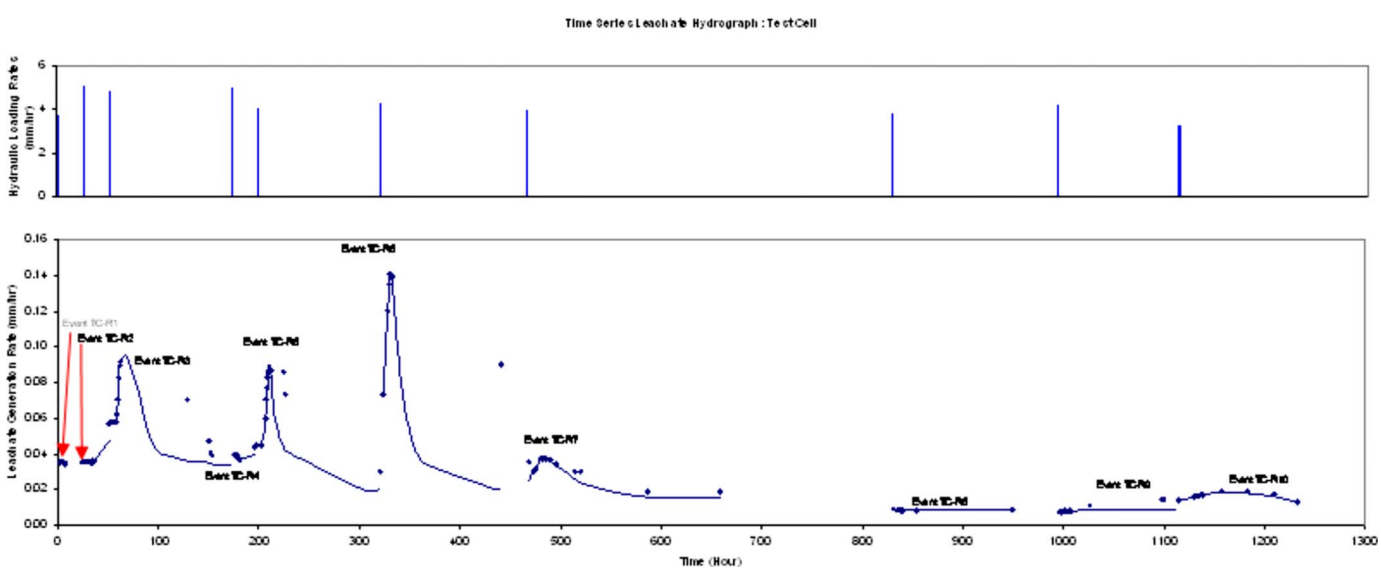


**Fig. 10.** Time-series leachate hydrographs—test cell



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**Table 7.** Leachate Generation Potential (LGP) and Peak Leachate Factor (PLF)—Test Cell

| Event | LGP | PLF |
|---|---|---|
| TC-R3 | 0.5950 | 2.04 |
| TC-R5 | 0.5938 | 3.16 |
| TC-R6 | 0.5934 | 3.77 |
| TC-R7 | 0.5931 | 1.71 |
| TC-R10 | 0.5935 | 1.2 |

pre-event moisture content is higher, there is a better chance of observing a leachate hydrograph and a high peak leachate generation rate and vice versa.

In practice, pre-event moisture contents are high in landfill cells located in tropical climates with frequent rainfall events and wet waste with high organic content, as well as in bioreactor landfill cells receiving recirculating leachate. Even in landfills located in temperate climates, summer months are known to produce frequent rainfall events increasing the moisture content of a waste cell rapidly. Such cells should exhibit leachate production patterns as observed in this research with defined leachate hydrographs and measurable peak leachate generation rates. In such situations, the use of a daily average leachate generation rate for design purposes should produce inferior LCRS and such systems will exhibit low leachate removal efficiencies.

The leachate hydrographs shown in Fig. 10 were used to determine LGP and PLF values for the field test cell. Only five of the leachate hydrographs with well defined parameter values were used for this purpose. The results are presented in Table 7.

As seen from Table 7, the LGP value did not change drastically during the study period, and was much lower than the LGP values observed with the lysimeter experiments. Nevertheless, the PLF value varied between a low of 1.2 to a high of 3.77. The maximum was observed with TC-R6, which followed TC-R5, another event showing a relatively high PLF value.

The PLF values of the test cell results were examined for their dependency on hydraulic loading rate. As previously discussed, events TC-R3, TC-R7, and TC-R10 had relatively longer time intervals between events than events TC-R5 and TC-R6. Therefore, events TC-R3, TC-R7, and TC-R10 could be categorized as a "dry" group of infiltration events, whereas the other two could be categorized as "wet" infiltration events. In the case of the "dry" infiltration events, there exists a linear relationship between PLF and hydraulic loading rate (Fig. 11). In the "wet" group, events TC-R4 and TC-R5 had a PLF value in the same range.

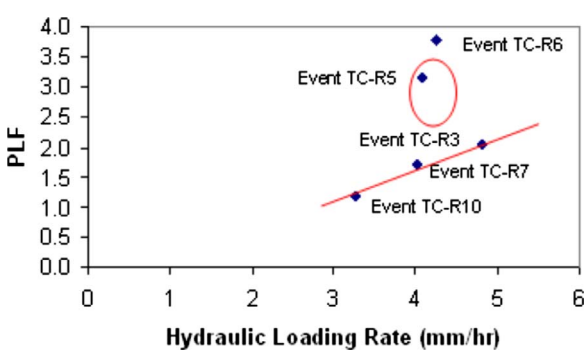


**Fig. 11.** Peak leachate factor (PLF) versus hydraulic loading rate—test cell

In comparison with lysimeter experiments, the test cell events recorded lower LGPs (about 0.6 compared with 0.8 to 1.5 for lysimeters). Production of leachate at lower LGP indicates higher incidence of "fingering" or short circuiting within the test cell. If significant short circuiting occurs, chances of observing leachate hydrographs will be lower. Nevertheless, several clearly defined leachate hydrographs were observed, which enabled determination of PLFs. The damping effect of the larger test cell compared with the smaller lysimeter was clearly evident with the relatively smaller PLFs observed. The high PLFs, 3.16 and 3.77, were observed with the two "wet" infiltration events, TC-R4 and TC-R5, respectively. This indicates the importance of determining PLF for landfills expecting "wet" infiltration events. A similar situation arises with flow through bioreactor landfills that are kept wet to increase biodegradation and rapid stabilization.

As indicated earlier, the LCRS in a landfill should be designed by assigning a safety factor to an average leachate generation rate predicted from one of the currently available models, such as the HELP model. The California landfill regulations require the use of twice the maximum anticipated daily leachate generation rate (Sharma and Lewis 1994), indicating the use of a safety factor of 2. However, this safety factor value appears to be assigned without a compelling technical basis. Current research provides such a technical basis. However, this research does not support applying a single safety factor value (or PLF) to a broad range of landfills.

Results from this research indicate the need to be selective in applying a safety factor in determining the design leachate generation or impingement rate. As such, the application of PLF as a safety factor in designing LCRS of landfills subjected to less frequent infiltration events, via rainfall or leachate recirculation, may not be useful. As indicated in Table 7, the PLF values associated with the infiltration events TC-R3, TC-R7, and TC-R10 ranged from 1.2 to 2. A safety factor of 2 as stipulated by the California landfill regulations would be sufficient for landfill cells subjected to conditions similar to TC-R3, TC-R7, and TC-R10. However, for landfill cells subjected to conditions similar to TC-R5 and TC-R6, a higher safety factor (PLFs between 3 and 4) should be used. In this case, a safety factor of 2 as stipulated by the California landfill regulations would cause an underdesign of the LCRS. Therefore, the application of PLF in designing a LCRS for a landfill subjected to frequent infiltration events, whether frequent rainfall events or continuous leachate recirculation, is critical.

In tropical climates, landfills are usually subjected to frequent rainfall events (almost daily) during the rainy seasons with high intensity and over long periods. It is critical that landfill designers adopt a sufficiently high safety factor to account for the peak leachate generation rates resulting from the high hydraulic loadings during rainy seasons. The maximum PLF generated from the test cell experiments was approximately 4. The PLF of tropical landfills can easily exceed the PLF of the test cell. The hydrological behavior of bioreactor landfills would be similar, where continuous leachate recirculation resembles that of landfills located in tropical climates. Therefore, it is also important to generate and apply PLFs in the design of LCRSs for bioreactor landfills as well.

## Conclusions

This paper presented a rational method to account for fluctuations in leachate generation by incorporating a correction factor to leachate generation rates determined from traditional techniques.



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The correction factor known as the peak leachate factor (PLF) is developed from landfill lysimeter and pilot scale test cell studies. It is defined as the ratio between peak and average daily leachate generation rate, and is proposed to be used in conjunction with the output from a leachate generation model such as the hydrologic evaluation of landfill performance (HELP) model to generate a design leachate generation rate.

The results indicated that various categories of landfill cells will require the adoption of PLFs depending on the type of waste landfilled and climatic conditions and operational characteristics of the landfills. The wet landfills, such as the ones located in tropical climates and bioreactor landfills may require the use of a PLF exceeding 4, the maximum value observed in field scale test cell studies. On the other hand, dry landfills could be designed with a PLF of 2, a value recommended by the California landfill regulations.

## Acknowledgments

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## References

Beaven, R. P., and Powrie, W. (1995). "Hydrological and geotechnical properties of refuse using a large scale compression cell." *Proc., Sardinia, '95, 5th Int. Landfill Symp.*, 745–760.

Bleiker, D. E., Farquhar, G., and McBean, E. (1995). "Landfill settlement and the impact on site capacity and refuse hydraulic conductivity." *Waste Manage. Res.*, 13, 533–554.

Campbell, D. J. V. (1982). "Absorptive capacity of refuse—Harwell research." *Harwell Landfill Leachate Symp.*

Chen, T.-H., and Chynoweth, D. P. (1995). "Hydraulic conductivity of compacted municipal solid waste." *Bioresour. Technol.*, 51, 205–212.

City of Calgary. (1999). "Design drawings for East Calgary landfill test cells (Micro No. 420-2306-000)." Solid Waste Services Division, Engineering and Environmental Services, City of Calgary.

Espinace Abarzúa, R., and Schiappacasse Dasati, C. (1997). "Project brief: Applicación de Líquidos Lixiviados a Residuos Sólidos Urbanos, Evaluación del Effecto en la Velocidad de los Procesos de Degradación, y en los Asientos." Dept. of Construction Engineering, Catholic Univ. of Valparaíso, Valparaíso, Chile.

Hillel, D. (1971). *Soil and water: Physical principles and processes*, Academic, New York.

Korfiatis, G. P., and Demetracopoulos, A. C. (1986). "Flow characteristics of landfill leachate collection systems and liners." *J. Environ. Eng.*, 112(3), 538–550.

Lambe, T. W. (1951). *Soil testing for engineers*, Wiley, New York.

McEnroe, B. M. (1989). "Steady drainage of landfill covers and bottom liners." *J. Environ. Eng.*, 115(6), 1114–1122.

Perera, L. A. K. (2001). "Gas migration model for sanitary landfill cover systems." Ph.D. thesis, Dept. of Civil Engineering, Univ. of Calgary, Calgary, Alberta, Canada.

Powrie, W., and Beaven, R. (1998). "Hydraulic conductivity of waste—Current research and implications for leachate management." *Wastes Management*, November, 22–23.

Sharma, H. D., and Lewis, S. P. (1994). *Waste containment systems, waste stabilizations and landfills—Design and evaluation*, Wiley, New York.

United States Environmental Protection Agency (USEPA). (1999). "Draft guide for industrial waste management Tier 3: Comprehensive risk assessment." *EPA 530-R-99-001*, ⟨http://www.epa.gov/industrialwaste/guide/chap7a-a.pdf⟩.



Pract. Period. Hazard. Toxic Radioact. Waste Manage. 2009.13:156-164.

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RECLAMATION

*Managing Water in the West*

***Report DSO-09-01***

# Physical Properties of Plastic Pipe Used in Reclamation Toe Drains

**Dam Safety Technology Development Program**

**U.S. Department of the Interior**
**Bureau of Reclamation**
**Technical Service Center**
**Denver, Colorado**

**September 2009**

REFERENCE 5

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# Physical Properties of Plastic Pipe Used in Reclamation Toe Drains

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# Executive Summary

In the 1980s, the Bureau of Reclamation (Reclamation) began using perforated plastic pipe for toe drains in dams. Corrugated high density polyethylene (HDPE) has been used most often, but other plastic pipe options include corrugated polyvinyl chloride (PVC), solid-wall PVC, and solid-wall HDPE. In 1999, Reclamation began video inspection of toe drains and found that about one-half of our plastic toe drains suffered from areas of serious deformation or collapse (Cooper, 2005). Concerns included whether all types of plastic pipe had sufficient strength and whether perforation patterns led to premature failure.

This study compares the strength and failure modes of perforated and non-perforated plastic pipe. Six types of plastic pipe were tested for pipe stiffness, ultimate strength, and mode of failure to determine suitability for use in toe drains. The six types of pipe evaluated include:

Single-wall corrugated HDPE drainpipe
Double-wall corrugated HDPE drainpipe
Double-wall corrugated PVC drainpipe
Solid-wall HDPE pressure pipe
Solid-wall PVC pressure pipe
Solid-wall PVC drainpipe[1]

Pipe strengths for perforated and non-perforated pipe were compared to published values. Pipe diameters ranged from 6 to 24 inches (although larger diameters are also commonly used). The manufacturer's standard perforation patterns were selected for testing.

## Conclusions

1. All the pipe exceeded the manufacturer's published values for pipe stiffness.

2. For corrugated pipe, the perforated pipe was just as strong as the non-perforated pipe. The reason for this high retained strength is that corrugated pipe gets its strength from the corrugation ribs, while the perforations are located in the corrugation valleys.

[1] For this report, solid-wall PVC drainpipe is non-pressurized drain-waste-vent (DWV) pipe, not to be confused with Reclamation terminology for perforated toe drainpipe for embankment dams.

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3. For solid-wall pipe, the perforated pipe was significantly weaker than the non-perforated pipe. The strength reduction was directly proportional to the percent open area (POA). Solid-wall PVC pipe (drainpipe and pressure pipe) showed a 2.5 percent decrease in pipe stiffness for every 1 percent open area. Solid-wall HDPE pipe lost more strength, with a 15.0 percent decrease in pipe stiffness for every 1 percent open area.

4. *Mode of Failure* – For all pipe tested, the presence and location of perforations had no influence on the mode or location of the pipe failure.

5. Both corrugated pipe options (HDPE and PVC) have significantly less strength (lower pipe stiffness) than both the solid-wall pressure pipe options (HDPE and PVC).

6. Lower strength corrugated plastic pipe relies greatly on support from the compacted backfill to resist deformation. Therefore, proper compaction and backfill support are critical for corrugated pipe.

7. *Brittleness* – Although not originally part of this study, brittleness proved to be an issue for some of the larger-diameter double-wall corrugated PVC pipe. This brittle behavior appears similar to field damage issues seen with thin-walled PVC pipe.

8. *Perforations* – Because of flow characteristics through perforations, fewer large holes are preferred for high-flow applications, while numerous slots (or smaller holes) are acceptable for low-flow applications.

9. *Joints* – All pipe manufacturers offer some type of satisfactory joint, with some joints easier to assemble and more robust than others. Since joints for perforated pipe need not be watertight, some manufacturers also offer a “soil-tight” joint that is appropriate for drainage applications.

10. *Recommendations* – Toe drains are critical to the safe operation of embankment dams. Toe drains frequently have deep burial where they would be difficult to access or replace. Trouble-free operation is essential, and small additional costs at the time of construction are easily justified. Proper installation of plastic pipe should be verified with closed circuit television inspection. Pipe recommendations for critical toe drain applications are shown in table ES-1.