B&L Eng. Calculation #5
Appendix 1C-5

Ref #2

Washington State
Department of Transportation

TECHNICAL MANUAL

Hydraulics Manual

M 23-03.01

March 2007

Environmental and Engineering Services
Hydraulics Office

2-5.4 Rainfall Intensity

After the appropriate storm frequency for the design has been determined (see Chapter 1) and the time of concentration has been calculated, the rainfall intensity can be calculated. Designers should never use a time of concentration that is less than 5 minutes for intensity calculations, even when the calculated time of concentration is less than 5 minutes. The 5-minute limit is based on two ideas:

1. Shorter times give unrealistic intensities. Many IDF curves are constructed from curve smoothing equations and not based on actual data collected at intervals shorter than 15 to 30 minutes. To make the curves shorter, involves extrapolation, which is not reliable.
2. It takes time for rainfall to generate into runoff within a defined basin, thus it would not be realistic to have less than 5 minutes for a time of concentration.

It should be noted that the rainfall intensity at any given time is the average of the most intense period enveloped by the time of concentration and is not the instantaneous rainfall. Equation 2-4 is the equation for calculating rainfall intensity.

$$I = \frac{m}{(T_C)^n} \qquad (2\text{-}4)$$

Where: I = Rainfall intensity in inches per hour (millimeters per hour)
T_C = Time of concentration in minutes
m & n = Coefficents in dimensionless units (Figures 2-5.4A & 2-5.4B)

The coefficients (m and n) have been determined for all major cities for the 2-, 5-, 10-, 25-, 50-, and 100 year mean recurrence intervals (MRI). The coefficients listed are accurate from 5-minute durations to 1,440-minute durations (24 hours). These equations were developed from the 1973 National Oceanic and Atmospheric Administration Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume IX-Washington.

With the Region Hydraulic Engineers assistance, the designer should interpolate between the two or three nearest cities listed in the tables when working on a project that is in a location not listed on the table. If the designer must do an analysis with a T_c greater than 1,440 minutes, the rational method should not be used.

Location	2-Year MRI		5-Year MRI		10-Year MRI		25-Year MRI		50-Year MRI		100-Year MRI	
	m	n	m	n	m	n	m	n	m	n	m	n
Aberdeen and Hoquiam	5.10	0.488	6.22	0.488	7.06	0.487	8.17	0.487	9.02	0.487	9.86	0.487
Bellingham	4.29	0.549	5.59	0.555	6.59	0.559	7.90	0.562	8.89	0.563	9.88	0.565
Bremerton	3.79	0.480	4.84	0.487	5.63	0.490	6.68	0.494	7.47	0.496	8.26	0.498
Centralia and Chehalis	3.63	0.506	4.85	0.518	5.76	0.524	7.00	0.530	7.92	0.533	8.86	0.537
Clarkston and Colfax	5.02	0.628	6.84	0.633	8.24	0.635	10.07	0.638	11.45	0.639	12.81	0.639
Colville	3.48	0.558	5.44	0.593	6.98	0.610	9.07	0.626	10.65	0.635	12.26	0.642
Ellensburg	2.89	0.590	5.18	0.631	7.00	0.649	9.43	0.664	11.30	0.672	13.18	0.678
Everett	3.69	0.556	5.20	0.570	6.31	0.575	7.83	0.582	8.96	0.585	10.07	0.586
Forks	4.19	0.410	5.12	0.412	5.84	0.413	6.76	0.414	7.47	0.415	8.18	0.416
Hoffstadt Cr. (SR 504)	3.96	0.448	5.21	0.462	6.16	0.469	7.44	0.476	8.41	0.480	9.38	0.484
Hoodsport	4.47	0.428	5.44	0.428	6.17	0.427	7.15	0.428	7.88	0.428	8.62	0.428
Kelso and Longview	4.25	0.507	5.50	0.515	6.45	0.509	7.74	0.524	8.70	0.526	9.67	0.529
Leavenworth	3.04	0.530	4.12	0.542	5.62	0.575	7.94	0.594	9.75	0.606	11.08	0.611
Metaline Falls	3.36	0.527	4.90	0.553	6.09	0.566	7.45	0.570	9.29	0.592	10.45	0.591
Moses Lake	2.61	0.583	5.05	0.634	6.99	0.655	9.58	0.671	11.61	0.681	13.63	0.688
Mt. Vernon	3.92	0.542	5.25	0.552	6.26	0.557	7.59	0.561	8.60	0.564	9.63	0.567
Naselle	4.57	0.432	5.67	0.441	6.14	0.432	7.47	0.443	8.05	0.440	8.91	0.436
Olympia	3.82	0.466	4.86	0.472	5.62	0.474	6.63	0.477	7.40	0.478	8.17	0.480
Omak	3.04	0.583	5.06	0.618	6.63	0.633	8.74	0.647	10.35	0.654	11.97	0.660
Pasco and Kennewick	2.89	0.590	5.18	0.631	7.00	0.649	9.43	0.664	11.30	0.672	13.18	0.678
Port Angeles	4.31	0.530	5.42	0.531	6.25	0.531	7.37	0.532	8.19	0.532	9.03	0.532
Poulsbo	3.83	0.506	4.98	0.513	5.85	0.516	7.00	0.519	7.86	0.521	8.74	0.523
Queets	4.26	0.422	5.18	0.423	5.87	0.423	6.79	0.423	7.48	0.423	8.18	0.424
Seattle	3.56	0.515	4.83	0.531	5.62	0.530	6.89	0.539	7.88	0.545	8.75	0.5454
Sequim	3.50	0.551	5.01	0.569	6.16	0.577	7.69	0.585	8.88	0.590	10.04	0.593
Snoqualmie Pass	3.61	0.417	4.81	0.435	6.56	0.459	7.72	0.459	8.78	0.461	10.21	0.476
Spokane	3.47	0.556	5.43	0.591	6.98	0.609	9.09	0.626	10.68	0.635	12.33	0.643
Stevens Pass	4.73	0.462	6.09	0.470	8.19	0.500	8.53	0.484	10.61	0.499	12.45	0.513
Tacoma	3.57	0.516	4.78	0.527	5.70	0.533	6.93	0.539	7.86	0.542	8.79	0.545
Vancouver	2.92	0.477	4.05	0.496	4.92	0.506	6.06	0.515	6.95	0.520	7.82	0.525
Walla Walla	3.33	0.569	5.54	0.609	7.30	0.627	9.67	0.645	11.45	0.653	13.28	0.660
Wenatchee	3.15	0.535	4.88	0.566	6.19	0.579	7.94	0.592	9.32	0.600	10.68	0.605
Yakima	3.86	0.608	5.86	0.633	7.37	0.644	9.40	0.654	10.93	0.659	12.47	0.663

Index to Rainfall Coefficients (English Units)

Figure 2-5.4A

2. Retention Ponds – When retention ponds are located near the roadway, the emergency spillway should be located outside of any snow storage areas that could block overflow passage, or an alternative flow route should be designated.

3. Frozen Ground - Frozen Ground coupled with snowmelt or rain on snow can cause unusually adverse conditions. These combined sources of runoff are generally reflected in the USGS regression equations as well as in the historic gauge records. No corrections or adjustments typically need to be made to these hydrology methods for frozen ground or snowmelt. For smaller basins, the SBUH and Rational methods are typically used to determine peak volume and peak runoff rates. The CN value for the SBUH method, and the runoff coefficient for the Rational method do not need to be increased to account for frozen ground in snowy or frozen areas as consideration has been given to this in the normal precipitation amounts and in deriving the snowmelt equation.

2-5 The Rational Method

2-5.1 General

The rational method is used to predict peak flows for small drainage areas, which can be either natural or developed. The rational method can be used for culvert design, pavement drainage design, storm drain design, and some eastern Washington stormwater facility design. The greatest accuracy is obtained for areas smaller than 100 acres (40 hectares) and for developed conditions with large areas of impervious surface (e.g., pavement, roof tops, etc.). Basins up to 1,000 acres (400 hectares) may be evaluated using the rational formula; however, results for large basins often do not properly account for effects of infiltration and thus are less accurate. Designers should never perform a rational method analysis on a basin that is larger than the lower limit specified for the USGS regression equations since the USGS regression equations will yield a more accurate flow prediction for that size of basin.

The formula for the rational method is:

$$Q = \frac{CIA}{K_C} \qquad (2\text{-}1)$$

Where:
- Q = Runoff in cubic feet per second (cubic meters per second)
- C = Runoff coefficient in dimensionless units
- I = Rainfall intensity in inches per hour (millimeters per hour)
- A = Drainage area in acres (hectares)
- K_C = Conversion factor of 1 for English (360 for metric units)

When several subareas within a drainage basin have different runoff coefficients, the rational formula can be modified as follows:

$$Q = \frac{I\Sigma CA}{K_C} \qquad (2\text{-}1a)$$

Where: $\sum CA = C_1 \times A_1 + C_2 \times A_2 + \ldots C_n \times A_n$

Hydrologic information calculated by the rational method should be submitted on DOT Form 235-009 (see Figure 2-5.1). This format contains all the required input information as well as the resulting discharge. The description of each area should be identified by name or stationing so that the reviewer may easily locate each area.

2-5.2 Runoff Coefficients

The runoff coefficient "C" represents the percentage of rainfall that becomes runoff. The rational method implies that this ratio is fixed for a given drainage basin. In reality, the coefficient may vary with respect to prior wetting and seasonal conditions. The use of an average coefficient for various surface types is quite common and it is assumed to stay constant through the duration of the rainstorm.

When considering frozen ground, designers should review section 2-4.2 number 3 of this manual.

In a high growth rate area, runoff factors should be projected that will be characteristic of developed conditions 20 years after construction of the project. Even though local storm water practices (where they exist) may reduce potential increases in runoff, prudent engineering should still make allowances for predictable growth patterns.

The coefficients in Figure 2-5.2 are applicable for peak storms of 10-year frequency. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. Generally, when designing for a 25-year frequency, the coefficient should be increased by 10 percent; when designing for a 50-year frequency, the coefficient should be increased by 20 percent; and when designing for a 100-year frequency, the coefficient should be increased by 25 percent. The runoff coefficient should not be increased above 0.95, unless approved by the Regional Hydraulics Engineer. Higher values may be appropriate for steeply sloped areas and/or longer return periods, because in these cases infiltration and other losses have a proportionally smaller effect on runoff.

Type of Cover	Flat	Rolling 2%-10%	Hilly Over 10%
Pavement and Roofs	0.90	0.90	0.90
Earth Shoulders	0.50	0.50	0.50
Drives and Walks	0.75	0.80	0.85
Gravel Pavement	0.50	0.55	0.60
City Business Areas	0.80	0.85	0.85
Suburban Residential	0.25	0.35	0.40
Single Family Residential	0.30	0.40	0.50
Multi Units, Detached	0.40	0.50	0.60
Multi Units, Attached	0.60	0.65	0.70
Lawns, Very Sandy Soil	0.05	0.07	0.10
Lawns, Sandy Soil	0.10	0.15	0.20
Lawns, Heavy Soil	0.17	0.22	0.35
Grass Shoulders	0.25	0.25	0.25
Side Slopes, Earth	0.60	0.60	0.60
Side Slopes, Turf	0.30	0.30	0.30
Median Areas, Turf	0.25	0.30	0.30
Cultivated Land, Clay and Loam	0.50	0.55	0.60
Cultivated Land, Sand and Gravel	0.25	0.30	0.35
Industrial Areas, Light	0.50	0.70	0.80
Industrial Areas, Heavy	0.60	0.80	0.90
Parks and Cemeteries	0.10	0.15	0.25
Playgrounds	0.20	0.25	0.30
Woodland and Forests	0.10	0.15	0.20
Meadows and Pasture Land	0.25	0.30	0.35
Pasture with Frozen Ground	0.40	0.45	0.50
Unimproved Areas	0.10	0.20	0.30

Runoff Coefficients for the Rational Method — 10-Year Return Frequency

Figure 2-5.2

x 1.25 to get 100-yr MRI

x 1.1 to get 25-yr MRI

2-5.3 Time of Concentration

If rainfall were applied at a constant rate over a drainage basin, it would eventually produce a constant peak rate of runoff. The amount of time that passes from the moment that the constant rainfall begins to the moment that the constant rate of runoff begins is called the time of concentration. This is the time required for the surface runoff to flow from the most hydraulically remote part of the drainage basin to the location of concern.

Actual precipitation does not fall at a constant rate. A precipitation event will begin with small rainfall intensity then, sometimes very quickly, build to peak intensity and eventually taper down to no rainfall. Because rainfall intensity is variable, the time of

DESIGN MEMORANDUM

Client: Hoyd | Snider — Sheet 1 Of 7

Project: B & L — Date: 5/11/09

Data For: Addendum I App 1C — Work Order:

Prepared By: Miao Zhang Checked By: DLS — File No:

Note: This form must be used for project calculations and original filed in project files

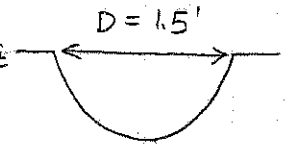
amec

Objective: Calculate the Capacity of 18"-diameter Half-pipe as Final Stormwater Drainage Ditch for the SW portion (total length ≅ 1000 ft)

Known/Assumed Parameters.

Manning's $n = 0.025$, upper range for Corrugated-interior HDPE pipe, Appendix A4-1, Reference # 1

Hydraulic Radius @ full flow $R = \frac{\pi(\frac{D}{2})^2/2}{\pi(\frac{D}{2})} = \frac{D}{4} = 0.375$ ft



Slope of pipe: $S = \frac{0.5}{300} = 0.0017$, from CAD drawing C-12, lower end of range

Calculations:

① 18"-diameter @ full flow

Chezy-Manning equation:

$$v = \left(\frac{1.49}{n}\right) R^{\frac{2}{3}} \sqrt{S}$$

$$= \left(\frac{1.49}{0.025}\right)(0.375)^{2/3}\sqrt{(0.0017)}$$

$$= 1.26 \text{ ft/s}$$

$$Q = v \cdot A = (1.26)\pi\left(\frac{1.5}{2}\right)^2\left(\frac{1}{2}\right) = 1.11 \text{ ft}^3/\text{s}$$ for full flow

less than peak discharge for 10-year or 25-year storms.

② Back calculate required slope for 18"-diameter half-pipe

trial & error in Excel $S^* = 0.008 \Leftrightarrow Q = 2.44$ ft³/s

$\Updownarrow$

8 ft drop over 1000 pipe, not possible!

DESIGN MEMORANDUM

Client:

Sheet 2 Of 7

Project: B & L

Date: 5/11/09

Data For: Addendum I App 1C

Work Order:

Prepared By: Miao Zhang Checked By: DLS

File No:

Note: This form must be used for project calculations and original filed in project files

③ Back calculate required half-pipe for the same slope

trial & error in Excel $D^* = 24$ inches

$\Updownarrow$

$Q^* = 2.40\ ft^3/s$ $V^* = 1.53\ ft/s$

④ Calculate Hydraulic Grade Line slope, pg 6-14, 6-15, Ref # 1 (see below)

Friction: $\frac{H_f}{L} = \left[\frac{2.15\, Q \cdot n}{D^{2.667}}\right]^2$

$$= \left(\frac{(2.15)(2.4)(0.035)}{2^{2.667}}\right)^2$$

$$= 0.0008\ ft/ft$$

Over 1000 ft, head drop $= 0.8$ ft

Minor Loss:

$$H_m = K \cdot \frac{(V^*)^2}{2g}, \quad \frac{(V^*)^2}{2g} = \frac{(1.53)^2}{(2)(32.2)} = 0.04\ ft$$

ft^2/s^2 ft/s^2

K = minor loss coefficient,

$$K = K_{entrance} + K_{exit} + K_{bend\ 45°}$$

$$= 1 + \frac{1}{2} + 0.47$$

$$\approx 2$$

$$\Rightarrow H_m = (2)(0.04\ ft) = 0.08\ ft$$

Total Head Loss $H_t = H_f + H_m = 0.88\ ft <$ Elevation Drop $(\sim 2\ ft)$ $\Rightarrow$ ok

Ref #1. WSDOT Hydraulics Manual, March 2007

DESIGN MEMORANDUM

amec

Client:

Sheet 3 Of 7

Project: B&L

Date: 5/11/09

Data For: Addendum I App 1C

Work Order:

Prepared By: Miao Zhang Checked By: DLS

File No:

Note: This form must be used for project calculations and original filed in project files

⑤ Calculate Safety Factor for 24-in-diameter half pipe

5.a. 10-year return interval

$$S.F. = \frac{Q^*}{Q_p\,(10\text{-yr})} = \frac{2.40\ ft^3/s}{1.48\ ft^3/s} = \boxed{1.62}$$

5.b 25-year return interval

$$S.F. = \frac{Q^*}{Q_p\,(25\text{-yr})} = \frac{2.40\ ft^3/s}{1.94\ ft^3/s} = \boxed{1.24}$$

DESIGN MEMORANDUM

Client: Floyd | Snider
Sheet 4 Of 7
Project: Bob Woodinsek Site
Date:
Data For: Addendum I Appendix 1C
Work Order:
Prepared By: RW Checked By: DLS
File No:

Note: This form must be used for project calculations and original filed in project files

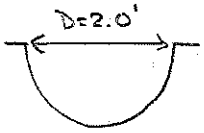
Objective: Calculate Capacity of 24"-diameter half-pipe as final stormwater drainage ditch for the East portion of the landfill (total length = ~ 1,052 ft)

Known/Assumed Parameters:

Manning's $n = 0.025$, upper range for corrugated-interior HDPE pipe, Appendix A4-1, Reference #1

Hydraulic Radius @ full flow

$$R = \frac{\frac{\pi (D/2)^2}{2}}{\pi \left(\frac{D}{2}\right)} = \frac{D}{4} = 0.5 \text{ ft}$$



Slope of Pipe $= \frac{1.5 \text{ ft}}{1{,}052 \text{ ft}} = 0.0014$, from CAD Drawing C-12

Calculations: ① 24"-diameter @ full flow

Chezy-Manning Equation

$$v = \frac{1.49}{n} R^{2/3} S^{1/2} = \left(\frac{1.49}{0.025}\right)\left(0.5^{2/3}\right)\left(0.0014\right)^{1/2}$$

$$= 1.40 \text{ ft/s}$$

$$Q = v \cdot A = \left(1.40 \frac{ft}{s}\right) \pi \left(\frac{2}{2}\right)^2 \left(\frac{1}{2}\right) = 2.21 \frac{ft^3}{s} \text{ for full flow}$$

Greater than peak flow for 25 year storm.

② Calculate Hydraulic Grade Line Slope pg 6-14, 6-15 WSDOT Hydraulics Manual, March 2007 (Ref #1)

Friction: $$\frac{H_f}{L} = \left[\frac{2.15\, Q \cdot n}{D^{2.667}}\right]^2$$

$$= \left[\frac{2.15 \cdot 2.21 \frac{ft^3}{s} \cdot 0.025}{2^{2.667}}\right]^2 = 0.00035 \frac{ft}{ft}$$

DESIGN MEMORANDUM

Client: Floyd | Snider Sheet 5 Of 7

Project: B&L Woodinaste Site Date:

Data For: Addendum I Appendix 1C Work Order:

Prepared By: CW Checked By: DLS File No:

Note: This form must be used for project calculations and original filed in project files

Over 1,052 ft, head drop = 0.37 ft

Minor Loss =

$$H_m = \frac{K (V^*)^2}{2g}, \quad \frac{(V^*)^2}{2g} = \frac{(1.40\ ft/s)^2}{2 \cdot 32.2\ \frac{ft}{s^2}} = 0.0304\ ft$$

K = minor loss coefficient

$K = K_{entrance} + K_{exit} + K_{bend\ 45°} + K_{bend\ 45°}$

$= 1 + 1/2 + (0.47)(2)$

≈ 2.4

$\Rightarrow H_m = (2.4)(0.0304\ ft) = 0.07296\ ft$

Total Head Loss $H_t = H_f + H_m$; $H_t = 0.37 + 0.07296 = 0.4\ ft$

$0.4 <$ Elevation Drop (~ 1.5 ft) $\Rightarrow$ OK

Ref #1 WSDOT Hydraulics Manual, March 2007

③ Calculate Safety Factor for 24-inch diameter half pipe

3.a. 10-year return interval

$$S.F. = \frac{Q^*}{Q_p\ (10\text{-yr})} = \frac{2.21\ ft^3/s}{1.50\ ft^3/s} = 1.47$$

3.b 25-year return interval

$$S.F. = \frac{Q^*}{Q_p\ (25\text{-yr})} = \frac{2.21\ ft^3/s}{1.97\ ft^3/s} = 1.12$$

DESIGN MEMORANDUM

Client: Floyd | Snider — Sheet 6 Of 7

Project: B+L Woodwaste Site — Date:

Data For: Addendum I Appendix 2C — Work Order:

Prepared By: RW — Checked By: DLS — File No:

Note: This form must be used for project calculations and original filed in project files

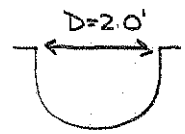
Objective: Calculate Capacity of 24"- diameter half-pipe as final stormwater drainage ditch for the north portion of the landfill (total length = ~ 503 ft)

Known / Assumed Parameters:

Manning's $n = 0.025$, upper range for corrugated-interior HDPE pipe (Appendix A1-1, Reference #1)

Hydraulic Radius @ full flow

$$R = \frac{\frac{\pi (D/2)^2}{2}}{\pi \left(\frac{D}{2}\right)} = \frac{D}{4} = 0.5 \text{ ft}$$



$$\text{Slope of Pipe} = \frac{1.0 \text{ ft}}{503 \text{ ft}} = 0.00199 \frac{\text{ft}}{\text{ft}}$$

Calculations: ① 24"- diameter @ full flow

Chezy-Manning Equation

$$v = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.025} (0.5)^{2/3} (0.00199)^{1/2}$$

$$= 1.67 \text{ ft/s}$$

$$Q = v \cdot A = \left(1.67 \frac{\text{ft}}{\text{s}}\right) \pi \left(\frac{2}{2}\right)^2 \left(\frac{1}{2}\right) = 2.63 \frac{\text{ft}^3}{\text{s}} \text{ for full flow}$$

Greater than peak for 25 year storm

② Calculate Hydraulic Grade Line Slope pg 6-14, 6-15 WSDOT Hydraulics Manual, March 2007 (Ref #1)

$$\text{Friction}: \frac{H_f}{L} = \left[\frac{2.15\, Q \cdot n}{D^{2.667}}\right]^2$$

$$= \left[\frac{2.15 \cdot (2.63 \text{ ft}^3/\text{s}) \cdot 0.025}{2^{2.667}}\right]^2 = 0.000495 \frac{\text{ft}}{\text{ft}}$$

DESIGN MEMORANDUM

Client: Floyd | Snider — Sheet 7 Of 7

Project: B+L Woodburnsite Site — Date:

Data For: Addendum I Appendix 1C — Work Order:

Prepared By: RW — Checked By: DLS — File No:

Note: This form must be used for project calculations and original filed in project files

Over 503 feet, head drop = 0.25 ft

Minor Loss =

$$H_m = \frac{K(V^*)^2}{2g}, \quad \frac{(V^*)^2}{2g} = \frac{(1.67\ ft/s)^2}{2 \cdot 32.2\ \frac{ft}{s^2}} = 0.043\ ft$$

K = minor loss coefficient

$$K = K_{entrance} + K_{exit} = 1 + 1/2 = 1.5$$

$$\Rightarrow H_m = (1.5)(0.043) = 0.0650\ ft$$

Total Head Loss $H_t = H_f + H_m$; $H_t = 0.25\ ft + 0.0650\ ft = 0.31\ ft$

0.31 ft < elevation drop (~1.0 ft) ⇒ OK

Ref #1 WSDOT Hydraulics Manual, March 2007

③ Calculate Safety Factor for 24-inch diameter half-pipe

3.a. 10-year return interval

$$SF = \frac{Q^*}{Q\ (10\text{-yr})} = \frac{2.63\ ft^3/s}{0.68\ ft^3/s} = 3.87$$

3.b. 25-year return interval

$$S.F = \frac{Q^*}{Q\ (25\text{yr})} = \frac{2.63\ ft^3/s}{0.89\ ft^3/s} = 2.96$$

B&L Eng. Calculation #5
Appendix 1C-5
Reference #1

Washington State
Department of Transportation

TECHNICAL MANUAL

Hydraulics Manual

M 23-03.01

March 2007

Environmental and Engineering Services
Hydraulics Office

Appendix 4-1 Manning's Roughness Coefficients (n)

I. Closed Conduits

A. Concrete pipe 0.010-0.011

B. Corrugated steel or Aluminum circular pipe or pipe-arch:

1. 2 2/3 × 1/2 in. Annular Corrugations, treated or untreated 0.022-0.027

2. 2 2/3 × 1/2 in. Helical Corrugations

a. Plain or Protective Treatments 1

(1) 18 inch diameter and below 0.013

(2) 24 inch diameter 0.015

(3) 36 inch diameter 0.018

(4) 48 inch diameter 0.021

(5) 60 inch diameter 0.022

(6) 72 inch diameter and above 0.024

b. Protective Treatments 2 or 4[1]

(1) 18 inch diameter and below 0.012

(2) 24 inch diameter 0.014

(3) 36 inch diameter 0.017

(4) 48 inch diameter 0.020

(5) 60 inch diameter 0.021

(6) 72 inch diameter and above 0.023

c. Protective Treatments 5 or 6[1]

(1) All diameters 0.012

3. 3 × 1 in. Annular Corrugations, treated or untreated 0.027-0.028

1. Treatments 3, 4 and 6 are no longer available and appear only for reference.

C. Steel or Aluminum Spiral Rib Pipe 0.012-0.013

D. Structural Plate Pipe and Plate Pipe Arches 0.033-0.037

E. Thermoplastic Pipe 0.012

1. Corrugated Polyethylene, HDPE 0.018-0.025
2. Profile wall polyvinyl chloride, PVC 0.009-0.011
3. Solid wall polyvinyl chloride, PVC 0.009-0.015

F. Cast-iron pipe, uncoated 0.013

G. Steel pipe 0.009-0.011

H. Vitrified clay pipe 0.012-0.014

I. Brick 0.014-0.017

J. Monolithic concrete:

1. Wood forms, rough 0.015-0.017
2. Wood forms, smooth 0.012-0.014
3. Steel forms 0.012-0.013

K. Cemented rubble masonry walls:

1. Concrete floor and top 0.017-0.022
2. Natural floor 0.019-0.025

L. Laminated treated wood 0.015-0.017

M. Vitrified clay liner plates 0.015

II. Open Channels, Lined (Straight Alignment)

A. Concrete, with surfaces as indicated:

1. Formed, no finish 0.013-0.017
2. Trowel finish 0.012-0.014
3. Float finish 0.013-0.015
4. Float finish, some gravel on bottom 0.015-0.017
5. Gunite, good section 0.016-0.019
6. Gunite, wavy section 0.018-0.022

As long as the HGL is lower than the rim elevation of the manhole or catch basin, the design is acceptable. If the HGL is higher than the rim elevation, flow will exit the storm drain and the design is unacceptable. The most common way to lower the HGL below the rim elevation is to lower the pipe inverts for one or more runs of the storm drain or increase the pipe diameter.

6-6.1 Friction Losses in Pipes

Head loss due to friction is a result of the kinetic energy lost as the flow passes through the pipe. The rougher the pipe surface is, the greater the head loss is going to be. Head loss from friction can be calculated with the following equation.

$$H_f = L\left[\frac{2.15Qn}{D^{2.667}}\right]^2 \text{ (English Units)} \qquad (6\text{-}4)$$

$$H_f = L\left[\frac{3.19Qn}{D^{2.667}}\right]^2 \text{ (Metric Units)}$$

Where:

H_f = Head loss due to friction in feet (meters)

L = Length of pipe in feet (meters)

Q = Flow in pipe in cfs (cms)

n = Manning's roughness coefficient (see Appendix 4-1)

D = Diameter of pipe in feet (meters)

6-6.2 Junction Entrance and Exit Losses

When flow enters a junction, it loses all of its velocity. As a result, there is an associated head loss equal to one velocity head. Then when the flow exits the junction and accelerates into the next pipe, there is another head loss equal to approximately half of one velocity head. These two head losses can be represented with the following equations (Metric and English units use the same equations).

$$H_e = \frac{V_2^{\,2}}{2g}$$

$$H_{ex} = 1.0\left(\frac{V^2}{2g} - \frac{V_d^{\,2}}{2g}\right) \approx \frac{V^2}{4g} \qquad (6\text{-}5)$$

Where H_e = head loss from junction entrance in feet (meters)

H_{ex} = head loss from junction exit in feet (meters)

V = flow velocity in pipe in feet per second (m/s)

V_d = channel velocity downstream of outlet in feet per second (m/s)

g = gravitational acceleration constant

6-6.3 Losses From Changes in Direction of Flow

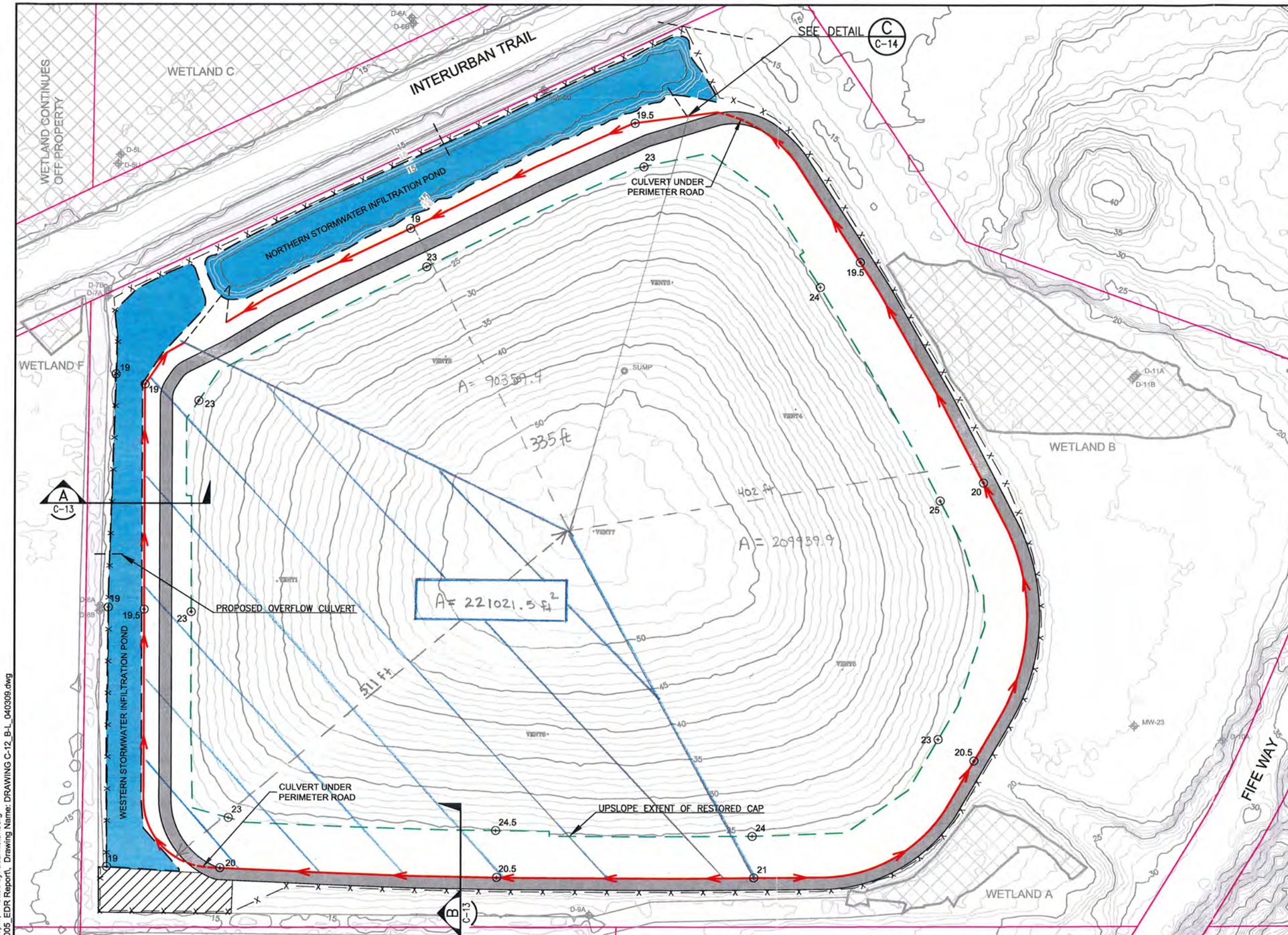
When flow changes direction inside of a junction, there is an associated head loss. The amount of head loss that will occur is dependent on how great the change is. As the angle between the inflow and outflow pipes increase, the amount of head loss increases. This head loss can be calculated with the following equation (metric and English units use the same equation).

$$H_b = K_b \frac{V^2}{2g} \qquad (6\text{-}6)$$

Where: H_b = Head loss from change in direction in feet (meters)

K_b = Head loss coefficient for change in direction, see below:

K_b	Angle of Change in Degrees
0.00	0
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90 and greater



EXPLANATION

- TAX PARCEL LINE
- FENCE LINE
- 12" DIAMETER CULVERT
- MW-13 MONITORING WELL LOCATION
- VENT3 VENT LOCATION
- AGRICULTURAL DITCH
- WETLAND
- FINAL STORMWATER COLLECTION DITCH
- 18" DIAMETER CULVERT
- STORMWATER FLOW DIRECTION
- INFILTRATION POND
- APPROXIMATE LOCATION OF COMPACTED STOCKPILE AREA
- FINAL PERIMETER ROAD
- 19.5 APPROXIMATE FINISHED ELEVATION (FT)

NOTES:

1. INCORPORATE AS MUCH OF EXCESS MATERIAL AS POSSIBLE INTO THE LANDFILL. GRADE AND COMPACT EXPOSED SUBGRADE.
2. CAP RESTORATION SHALL MEET THE EXISTING CAP AND EXTEND BEYOND THE BARRIER WALL. REUSE SALVAGED GEOSYNTHETICS MATERIAL TO THE EXTENT POSSIBLE, AND SALVAGED SOIL FROM SITE EARTHWORK TO COMPLETE THE CAP.
3. FINISH GRADE ON THE LANDFILL CAP SHALL BE A SMOOTH GRADE WITHOUT SUDDEN OR STEP CHANGES, EXCEPT NEAR THE ACCESS ROAD ON THE LANDFILL.
4. FINAL STORMWATER COLLECTION DITCH SHALL FOLLOW THE GRADE OF PERIMETER BERM AND BE CONNECTED TO THE THREE EXISTING CULVERTS TO THE NORTHERN INFILTRATION POND.
5. REMOVE GEOTEXTILE FROM THE PERIMETER FENCE AND DISPOSE OFFSITE.
6. INSTALL CHAIN LINK FENCE ALONG THE WEST BERM OF THE WEST INFILTRATION POND AND CONNECT WITH EXISTING PERIMETER FENCE.
7. REMOVE ALL EXCESS MATERIAL AND REMAINING DEBRIS FROM THE WORK AREA AND DISPOSE IN A SOLID WASTE LANDFILL IN ACCORDANCE WITH APPLICABLE REGULATIONS AND STANDARDS.
8. ALL DISTURBED AREAS SHALL BE HYDROSEEDED OR FINISHED AS APPROPRIATE UPON COMPLETION OF CONSTRUCTION ACTIVITIES. FILL USED IN THE CONTRACTOR STAGING AREA SHALL REMAIN.

DRAFT
CONSTRUCTION

0 50 100
APPROXIMATE SCALE IN FEET

REFERENCES:
PLANS

DATUM
HORIZONTAL: WASP-NAD83-S FEET
VERTICAL: NAVD88 FEET

NO.	REVISION	DATE	APRVD

DRAWN ??
DESIGNED ??
CHECKED ??
REVIEWED ??

FLOYD | SNIDER
AMEC Geomatrix

LANDFILL CAP RESTORATION PLAN
ADDENDUM 1

B&L WOODWASTE SITE
PIERCE COUNTY, WASHINGTON

DATE: 04/23/09
PROJECT NO.: 13488

DRAWING
C-12

Plot Date: 04/23/09 - 4:59pm, Plotted by: adam.stenberg
Drawing Path: S:\13488\005_EDR Report\, Drawing Name: DRAWING C-12_B-L_040309.dwg

DESIGN MEMORANDUM

Client: Floyd | Snider
Sheet 1 Of 9
Project: B&L Woodwaste Site
Date: 10/15/09
Data For: Phase 1 Part 1 Design
Work Order: 13488
Prepared By: RW/MZ Checked By: KT
File No:

Note: This form must be used for project calculations and original filed in project files

Objective Determine Seasonal Volumetric Flux Through The Sand Aquifer Beneath Woodwaste Landfill

- Using GIS analysis, average thickness of Upper Sand Aquifer is ~ 14.4 ft.
- Based upon CAD Drawing, width of landfill is ~ 860 feet (across groundwater flow direction)
- Assume $K = 50 \frac{ft}{d}$
- Use head values from Feb '09 (Sheet 9) to calculate wet season volumetric flux (Q_{wet})

where $Q_{wet} = K A \frac{H_{D-5U} - H_{MW-23}}{\Delta L}$

H_{D-5U} = Groundwater Elevation at D-5U (ft)
H_{MW-23} = Groundwater Elevation at MW-23 (ft)
ΔL = Distance between D-5U and MW-23 (ft) = 1100 ft
A = Cross Sectional Area Perpendicular to flow direction (ft^2)
K = hydraulic conductivity (ft/d)

$$Q_{wet} = \left(50 \frac{ft}{d}\right)\left(860 ft \cdot 14.4 ft\right)\left(\frac{17.56 - 13.28 ft}{1100 ft}\right)$$

$$= 2409 \ ft^3/d = \frac{2409 \ ft^3/d}{192.5 \frac{ft^3/d}{gpm}} = 12.5 \ gpm$$

- Use head values from Oct '08 (Sheet 8) to calculate dry volumetric flux (Q_{dry})

$$Q_{dry} = \left(50 ft\right)\left(860 ft \cdot 14.4 ft\right)\left(\frac{15.67 - 12.36}{1100 ft}\right)$$

$$= 1863 \ ft^3/d = 9.7 \ gpm$$

DESIGN MEMORANDUM

Client: Floyd/Snider
Sheet 2 Of 9
Project: B&L Woodwaste Site
Date: 6/15/09
Data For: Phase 1 Part 1 Design
Work Order: 13488
Prepared By: MZ Checked By: KT
File No:

Note: This form must be used for project calculations and original filed in project files

Seasonal Water Level Data

	MW-23	D-8A	D-54	ΔH MW-23 – D-54	ΔH MW-23 – D-8A
(dry) October '08	15.674	13.67	12.364	3.31	2.00
November '08	16.11	14.9	13.30	2.81	1.21
December '08	16.564	14.52	13.374	3.19	2.04
January '09	17.27	14.82	14.06	3.21	2.45
(wet) February '09	17.56	14.49	13.28	4.28	3.07

DESIGN MEMORANDUM

Client: Floyd | Snider Sheet 3 Of 9
Project: B&L Woodwaste Site Date: 6/15/09
Data For: Phase 1 Part 1 Design Work Order: 13488
Prepared By: MZ Checked By: KT File No:

Note: This form must be used for project calculations and original filed in project files

Objective: Estimate the required infiltration rates if Qwet is to be infiltrated through the infiltration ponds

- Assume half of the collected flow goes to the West Infiltration Pond

$$Q_w = \frac{2409\ ft^3/d}{2} = 1205\ ft^3/d$$

Bottom Area of pond, $A_w = 6342\ ft^2$

Required infiltration rate to infiltrate all the water (through pond bottom only)

$$I_w = \frac{Q_w}{A_w} = \frac{1205\ ft^3/d}{6342\ ft^2} = 0.19\ ft/d = 0.10\ in/hr$$

- Assume half of flow goes to the North Infiltration Pond

$$Q_n = \frac{2409\ ft^3/d}{2} = 1205\ ft^3/d$$

Bottom Area of Pond, $A_n = 15434\ ft^2$

Required infiltration rate

$$I_n = \frac{Q_n}{A_n} = \frac{1205\ ft^3/d}{15434\ ft^2} = 0.08\ ft/d = 0.04\ in/hr$$

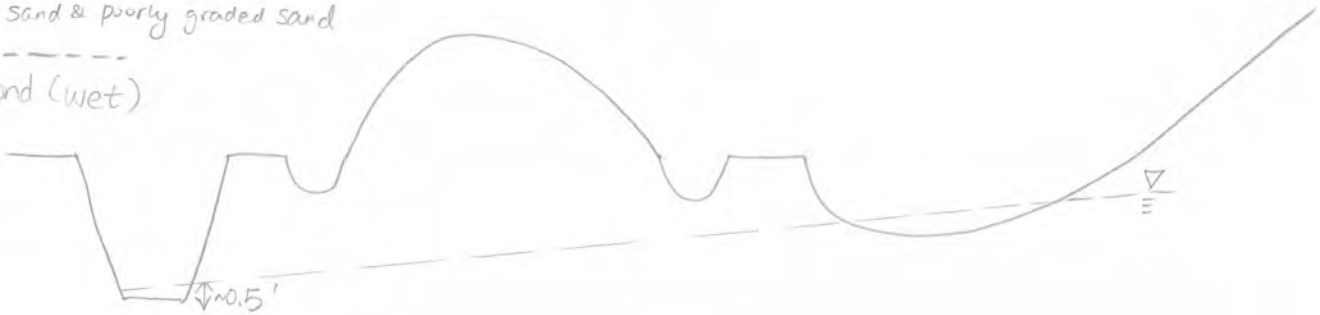
DESIGN MEMORANDUM

Client: Floyd | Snider
Sheet 4 Of 9
Project: B&L Woodwaste Site
Date: 6/15/09
Data For: Phase 1 Part 1 Design
Work Order: 13488
Prepared By: MZ
Checked By: KT
File No:

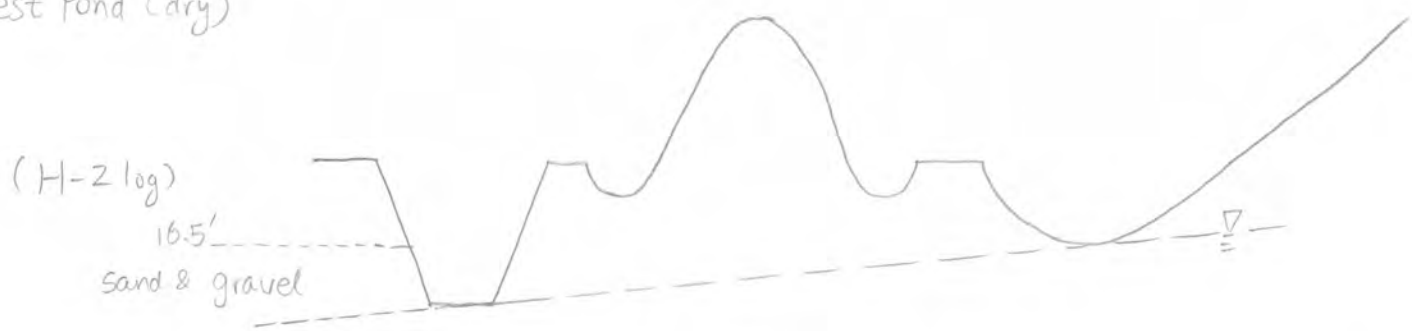
Scenario Note: This form must be used for project calculations and original filed in project files

① North Pond (dry)

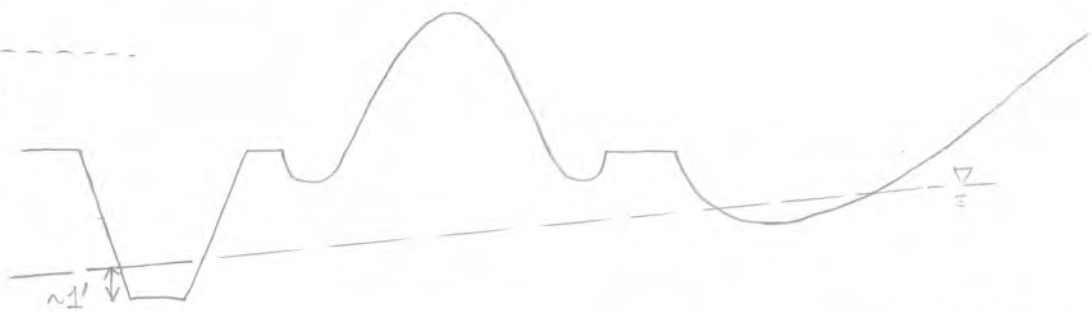
② North Pond (wet)



③ West Pond (dry)



④ West Pond (wet)



NOT TO SCALE

"Dry" = October '08 condition

"Wet" = February '09 condition

Water table based on data on sheet 2

DESIGN MEMORANDUM

Client: Floyd | Snider Sheet 5 Of 9
Project: B&L Woodwaste Site Date: 6/15/09
Data For: Phase 1 Part 1 Design Work Order: 13488
Prepared By: MZ Checked By: KT File No:

Note: This form must be used for project calculations and original filed in project files

△ Scenario 1 — North Pond, dry condition, water table is 1' below bottom

- Based on boring logs for H-6 and H-7,

  well-graded sand is between the pond bottom and water table.

- Infiltration rate for sand = 20 mm/hr
  = 0.8 in/hr

  [Irrigation Methods, UN Food and Agriculture Organization Ref # 1]

- Divide by a factor of 3 to account for that only 1/3 of pond bottom is underlied by well-graded sand.

$$I = \frac{0.8}{3} \text{ in/hr} = 0.26 \text{ in/hr} \gg I_{(n)} \ (0.04 \text{ in/hr}) \quad ✓$$

- If water table rises above pond bottom, see calculations below ⇒ $Q > Q_n$

△ Scenario 2 — North Pond, wet condition

- Water table is 0.5' above pond bottom
- The pond acts as a constant head boundary for downgradient area.
- Based on observed data and numerical model, flow direction is to the northwest
- $\boxed{\text{Assume there is no flooding from surface water}}$

  & water surface in the pond is at 16' (i.e. 3.5' of water)

- Use well PD-1B to estimate head gradient (distance = 650 ft)

$$i = (16' - 13.1')/(650 \text{ ft}) = 0.0045$$

- Use the same average aquifer thickness, b = 14.4 ft

  K = 50 ft/d

  Width of pond, w = 500 ft
  in west-east direction

DESIGN MEMORANDUM

Client: Floyd | Snider Sheet 6 Of 9
Project: B&L Woodwaste Site Date: 6/15/09
Data For: Phase 1 Part 1 Design Work Order: 13488
Prepared By: MZ Checked By: KT File No:

Note: This form must be used for project calculations and original filed in project files

Groundwater Flux $Q = K \cdot i \cdot w \cdot b$

$= (50\ ft/d)(0.0045)(500\ ft)(14.4\ ft)$

$= 1620\ ft^3/d \quad > Q_n$ ✓
$(1205\ ft^3/d)$

△ Scenario 3 – West Pond, dry condition

- Based on H-2, H-3, & H-4 logs, sand and gravel are below the pond.
- Water table is at pond bottom; therefore, the pond acts as a constant head boundary
- Based on observed data and model simulation, flow direction is to the west
- Take a point A due west of well D-8A (distance = 650 ft)

  Linearly interpolate head at A from PD-209A and SG-213 water levels in October '08 (sheet 8)

  $h_A = 11'$
- $\boxed{\text{Assume there is no flooding from surface water}}$

  & water surface in the pond is at 16'
- Gradient $i = \dfrac{(16' - 11')}{(650\ ft)} = 0.0077$
- $b = 14.4\ ft$
  $K = 50\ ft/d$
  $w = 600\ ft$, width of west pond in north-south direction

Groundwater Flux, $Q = K \cdot i \cdot wb = (50\ ft/d)(0.0077)(600\ ft)(14.4\ ft)$

$= 3323\ ft^3/d \quad > Q_w$ $(1205\ ft^3/d)$ ✓

DESIGN MEMORANDUM

Client: Floyd I Snider | Sheet 7 Of 9
Project: B&L Wood waste Site | Date: 6/15/09
Data For: Phase 1 Part 1 Design | Work Order: 13488
Prepared By: MZ | Checked By: KT | File No:

Note: This form must be used for project calculations and original filed in project files

△ Scenario 4 - West Pond, wet condition

- Same assumptions as scenario 3

- $h_A = 13.55'$ (Feburary'09 data; sheet 9)

- Gradient $i = \frac{(16' - 13.55')}{650\ ft} = 0.0038$

- $b = 14.4\ ft$
  $k = 50\ ft$
  $w = 600\ ft$

Groundwater Flux, $Q = K \cdot i \cdot w \cdot b = (50\ ft/d)(0.0038)(600\ ft)(14.4\ ft)$
$= 1642\ ft^3/d > Q_w$ ✓
$(1205\ ft^3/d)$

△ Conclusion:

The groundwater flux in the Upper Sand Aquifer beneath the Landfill can be infiltrated through the two infiltration ponds without overtopping, provided that surface water flow does not cause regional flooding and result in a "bath-tub" condition.