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Appendix H-7

Final Cover Soil Erosion Estimate

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Calculation Sheet

Client: <u>CWM Chemical Services, LLC</u>

Project Location: <u>Model City, New York</u>

Project: <u>RMU-2 Design Calculations</u> **Project No.: <u>B0023725.2009</u>**

Subject: <u>Appendix H-7: Final Cover Soil Erosion Estimate</u>

Prepared By: <u>JEM</u> **Date: <u>August 2009</u>**

Reviewed By: <u>JEM</u> **Date: <u>August 2009</u>**

Checked By: <u>BMS</u> **Date: <u>August 2009</u>**

<u>OBJECTIVE</u>:

Determine the estimated soil loss rate from RMU-2 for the final cover condition with established vegetation.

<u>REFERENCES</u>:

1. RMU-2 Permit Drawing No. 7 entitled "Top of Vegetative Cover Grades", ARCADIS, August 2009.

2. Revised Universal Soil Loss Equation Version 2.0 (RUSLE) computer software developed jointly by ARS and NRCS in West Lafayette, Indiana; Tucson, Arizona; Oxford, Mississippi; and Pullman, Washington, with special contributions made by the US Department of the Interior - Office of Surface Mines in extending the RUSLE to mining, construction, and reclaimed lands, May 2008.

3. User's Reference Guide for Revised Universal Soil Loss Equation (RUSLE) Version 2.0, George Foster, Research Hydraulic Engineer, National Sedimentation Laboratory, December 2004.

4. "Evaluating Cover Systems for Solid and Hazardous Waste (SW-867)," US Environmental Protection Agency, September 1980.

5. "New York State Standards and Specifications for Erosion and Sediment Control", New York State Department of Environmental Conservation, August 2005.

<u>ASSUMPTIONS</u>:

1. Final cover surface soil type is silty clay loam with K = 0.26.

2. Hydrologic Soil Group of final cover surface soil is "C."

3. General land use designated for RUSLE2 is highly disturbed land, long term vegetation, dense grass.

4. Final cover is established grass vegetation with rotary mower cover management.

5. Recommended maximum soil loss rate is 2 tons/acre/year (based on Reference 4).

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CALCULATIONS:

The estimated soil loss rate is calculated using RUSLE2 software for the worst-case uniform slope segment and for the worst-case complex (composite) slope segment as determined from Reference 1. The uniform slope consists of approximately 123 feet at 33%, beginning at coordinate location 8871N, 11,687E and extending to the west until it intercepts the toe of the final cover system at coordinate location 8,871N, 11,810E (based on the site coordinate system). The complex slope consists of six segments with a total length of approximately 400 feet and an average slope of 23% beginning on the landfill plateau at coordinate location 9,086N, 10,943E and extending to the west until it intercepts the toe of the final cover system at coordinate location 9,063N, 10,542E. Both slope locations are shown in Attachment 1.

RESULTS:

The average annual soil loss is estimated to be the following:

Worst-Case Uniform Slope – 1.7 tons/acre/year

Worst-Case Complex Slope – 0.83 tons/acre/year (average for entire slope)

Supporting RUSLE2 output for determination of the average annual soil loss is provided in Attachment 2.

Because of the complex mathematics employed by RUSLE2, it is not possible to simply multiply the relevant factors to obtain the average annual erosion rate as was the case with previous versions of RUSLE. However, the following is provided as a check of the soil loss for the uniform slope condition:

The RUSLE is as follows:

A=(R)(K)(LS)(C)(P)

where,

A = Average annual soil loss in tons per acre per year
R = 72 (rainfall-runoff erosivity factor as determined from Reference 5)
K = 0.26 (soil erodibility factor as determined from Reference 2)
LS = 5.70 (slope-length factor from Reference 2)
C = 0.018 (cover management factor as determined from Reference 2)
P = 1.0 (erosion control support practice factor as determined from Reference 2)

The estimated average annual rate of soil loss based on the above slope using the RUSLE is:

A = (72)(0.26)(5.70)(0.018)(1.0) = 1.9 tons/acre/year

SUMMARY:

The estimated soil loss rate from the RMU-2 final cover is 1.7 tons/acre/year or less under vegetated conditions. This is considered to be a conservative value, as it is based on the worst-case slope conditions. The estimated soil loss rate under vegetated conditions is less than the maximum rate of 2 tons/acre/year recommended by the USEPA.

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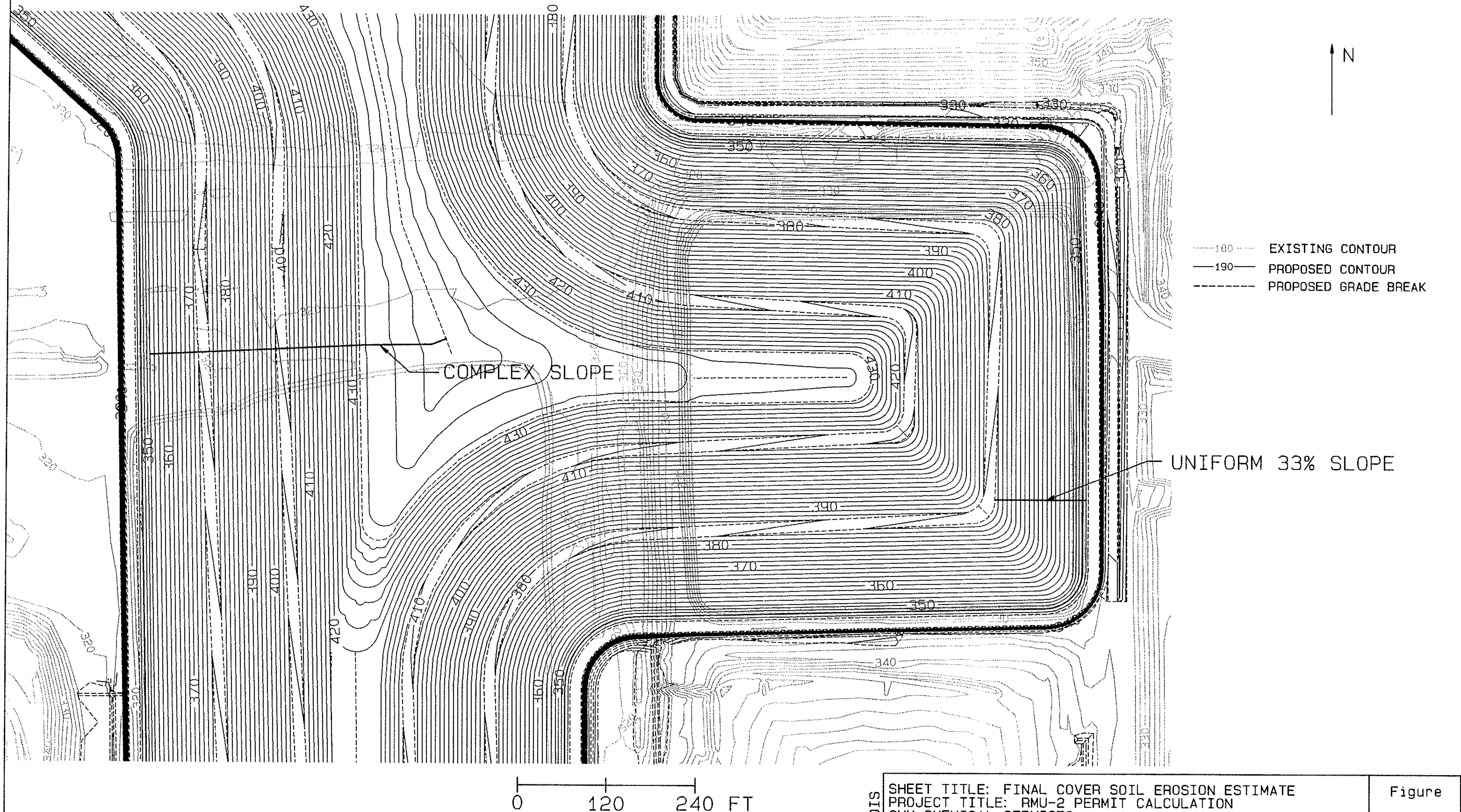
Attachment 1

Slope Location Map



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SHEET TITLE: FINAL COVER SOIL EROSION ESTIMATE
PROJECT TITLE: RMU-2 PERMIT CALCULATION
CWM CHEMICAL SERVICES
PROJECT LOCATION: MODEL CITY, NEW YORK
PROJECT NO: B0023725.2009
DATE: AUGUST 2009 BY: PTO FILE NO.: 23725.PRO

Figure 1

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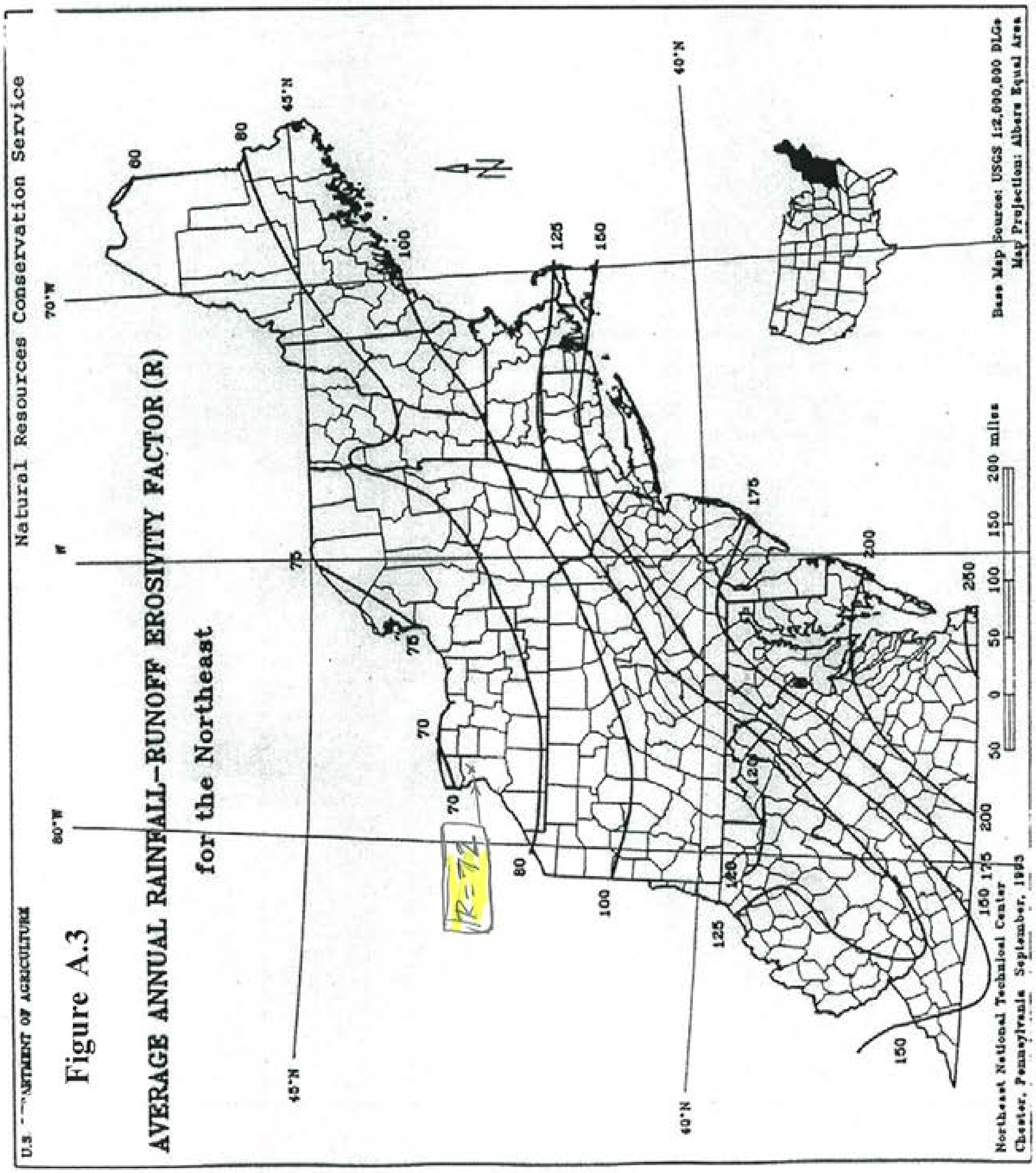
Attachment 2

Calculations


U.S. DEPARTMENT OF AGRICULTURE
Natural Resources Conservation Service
Figure A.3
AVERAGE ANNUAL RAINFALL-RUNOFF EROSIVITY FACTOR (R)
for the Northeast
80°W
75°W
70°W
45°N
40°N
N
60
70
75
80
100
125
150
175
200
250
R=72
50 0 50 100 150 200 miles
Northeast National Technical Center
Chester, Pennsylvania September, 1993
Base Map Source: USGS 1:2,000,000 DLGs
Map Projection: Albers Equal Area

RUSLE2 ARS Version May 20 2008 - [Climate: New York\Erie county average (Buffalo)*]

File Database Edit View Options Window Help

Auto update

How get erosivity distribution? Enter R & choose EI zone

R Factor, US 72.0

Standard EI distribution US_112

In Req area? No

How determine runoff? based on 10-yr 24-hr ppt

10-yr 24-hr rainfall, in. 3.6

10 year EI, US 34.4

Annual precip, in. 40

Monthly | Daily | Info

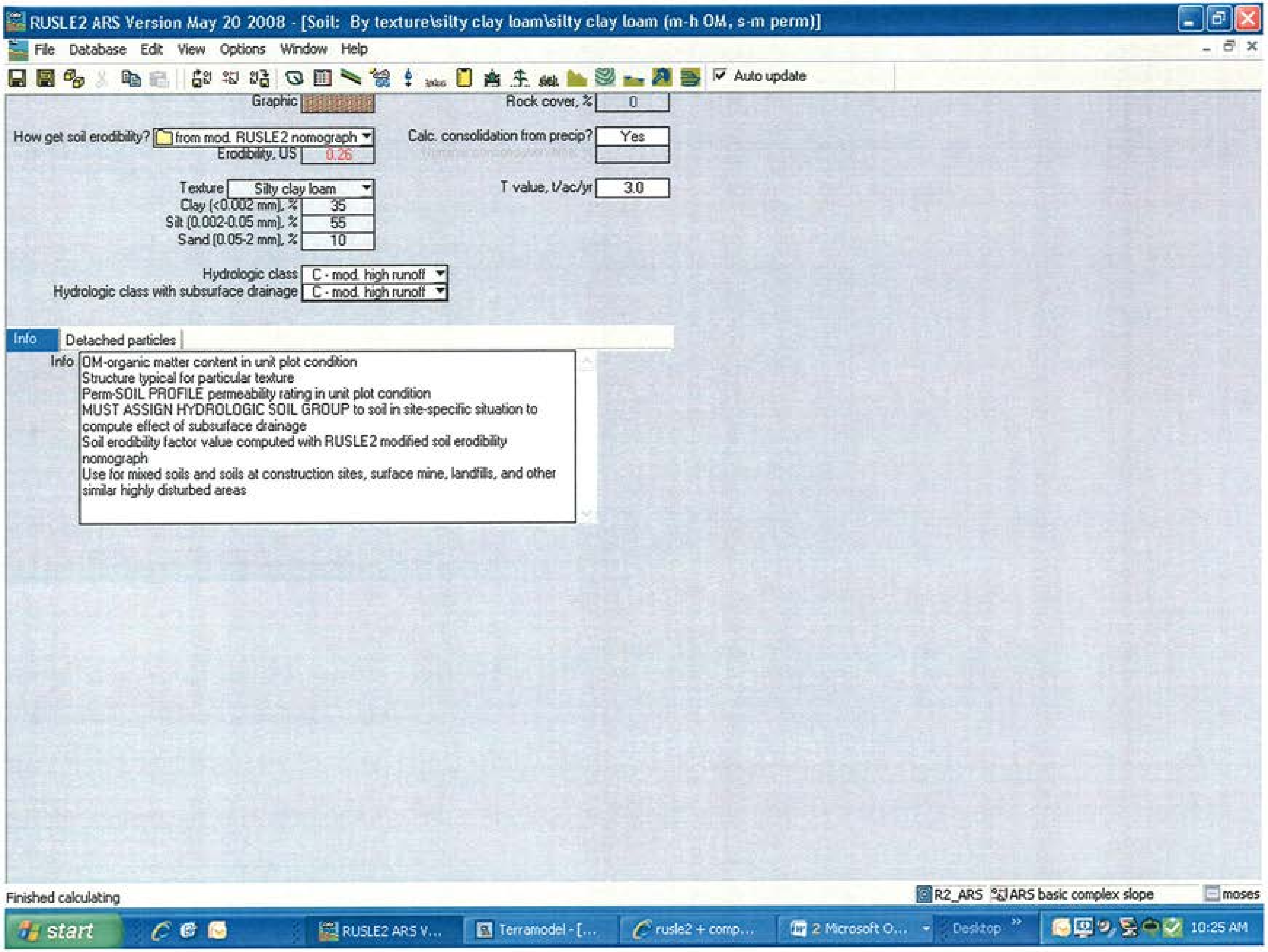
Monthly Climate

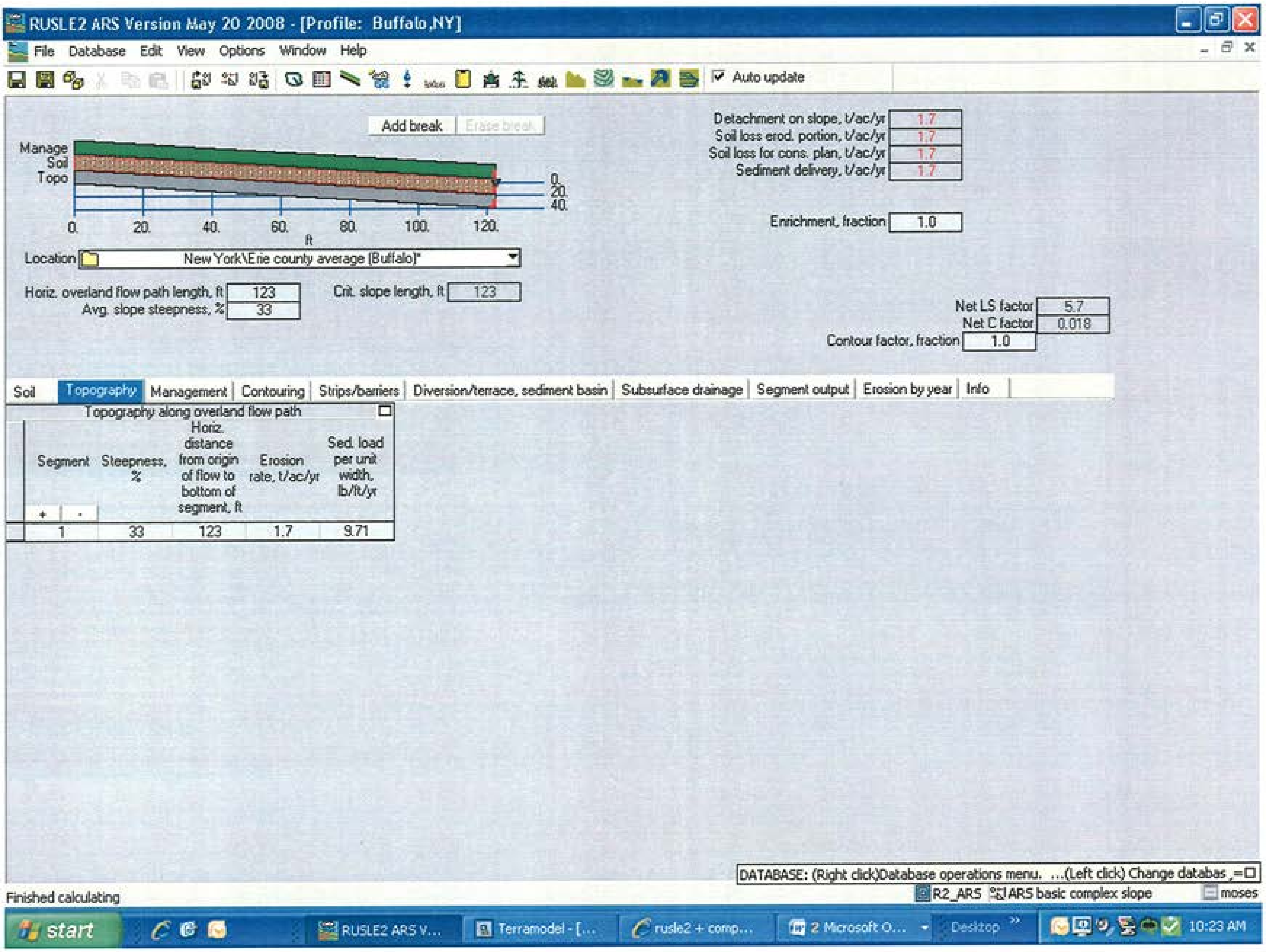
Month	Avg. temp., deg F	Month precip., in.	R values, US	Eros. density, US eros. / in. ppt
Jan	23	2.7	0	0
Feb	24	2.3	1.4	0.618
Mar	33	2.8	1.4	0.520
Apr	45	3.2	2.2	0.681
May	56	3.3	7.2	2.21
Jun	65	3.8	12	3.02
Jul	70	3.3	16	4.77
Aug	68	4.0	15	3.81
Sep	61	4.0	9.4	2.31
Oct	51	3.3	3.6	1.09
Nov	40	4.0	2.9	0.720
Dec	29	3.7	1.4	0.388

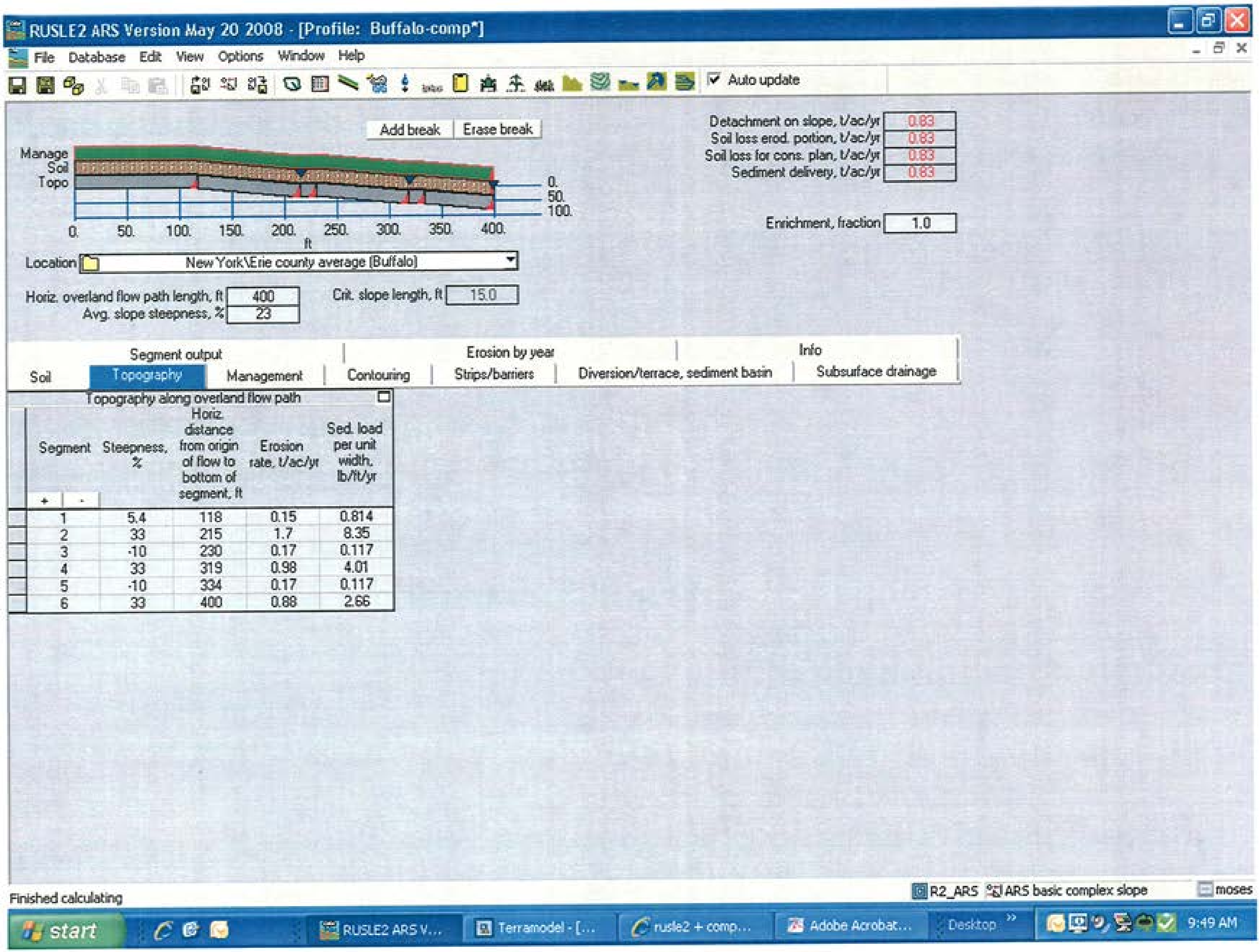
Finished calculating

R2_ARS ARS basic com Wednesday, July 08, 2009

start RUSLE2 ARS V... Terramodel - [... rusle2 + comp... 2 Microsoft O... Desktop 10:24 AM


RUSLE2 ARS Version May 20 2008 - [Soil: By texture\silty clay loam\silty clay loam (m-h OM, s-m perm)]
File Database Edit View Options Window Help
Auto update
Graphic
Rock cover, % 0
How get soil erodibility? from mod. RUSLE2 nomograph
Erodibility, US 0.26
Calc. consolidation from precip? Yes
Texture Silty clay loam
Clay (<0.002 mm), % 35
Silt (0.002-0.05 mm), % 55
Sand (0.05-2 mm), % 10
T value, t/ac/yr 3.0
Hydrologic class C - mod. high runoff
Hydrologic class with subsurface drainage C - mod. high runoff
Info
Detached particles
Info
OM-organic matter content in unit plot condition
Structure typical for particular texture
Perm-SOIL PROFILE permeability rating in unit plot condition
MUST ASSIGN HYDROLOGIC SOIL GROUP to soil in site-specific situation to compute effect of subsurface drainage
Soil erodibility factor value computed with RUSLE2 modified soil erodibility nomograph
Use for mixed soils and soils at construction sites, surface mine, landfills, and other similar highly disturbed areas
Finished calculating
R2_ARS
ARS basic complex slope
moses
start
RUSLE2 ARS V...
Terramodel - [...
rusle2 + comp...
2 Microsoft O...
Desktop
10:25 AM


RUSLE2 ARS Version May 20 2008 - [Profile: Buffalo,NY]
File Database Edit View Options Window Help
Auto update
Add break
Erase break
Manage
Soil
Topo
0. 20. 40. 60. 80. 100. 120.
ft
0.
20.
40.
Location
New York\Erie county average [Buffalo]*
Horiz. overland flow path length, ft 123
Avg. slope steepness, % 33
Crit. slope length, ft 123
Detachment on slope, t/ac/yr 1.7
Soil loss erod. portion, t/ac/yr 1.7
Soil loss for cons. plan, t/ac/yr 1.7
Sediment delivery, t/ac/yr 1.7
Enrichment, fraction 1.0
Net LS factor 5.7
Net C factor 0.018
Contour factor, fraction 1.0
Soil
Topography
Management
Contouring
Strips/barriers
Diversion/terrace, sediment basin
Subsurface drainage
Segment output
Erosion by year
Info
Topography along overland flow path
Segment
Steepness, %
Horiz. distance from origin of flow to bottom of segment, ft
Erosion rate, t/ac/yr
Sed. load per unit width, lb/ft/yr
1 33 123 1.7 9.71
DATABASE: (Right click)Database operations menu. ...(Left click) Change databas
R2_ARS
ARS basic complex slope
moses
Finished calculating
start
RUSLE2 ARS V...
Terramodel - [...
rusle2 + comp...
2 Microsoft O...
Desktop
10:23 AM



Segment	Steepness, %	Horiz. distance from origin of flow to bottom of segment, ft	Erosion rate, t/ac/yr	Sed. load per unit width, lb/ft/yr
1	5.4	118	0.15	0.814
2	33	215	1.7	8.35
3	-10	230	0.17	0.117
4	33	319	0.98	4.01
5	-10	334	0.17	0.117
6	33	400	0.88	2.66

Appendix H-8

Stormwater Retention Basin
Capacity Analysis

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Calculation Sheet

Client: CWM Chemical Services, LLC

Project Location: Model City, New York

Project: RMU-2 Design Calculations **Project No.: B0023725.2011**

Subject: Appendix H-8: Stormwater Retention Basin Capacity Analysis

Prepared By: JEM **Date: February 2013**

Reviewed By: JEM **Date: February 2013**

Checked By: BMS **Date: February 2013**

OBJECTIVE:

Determine the required storage volume for the existing V01, V02, V04, and V05 stormwater retention basins to contain runoff from the 25-year, 24-hour design storm and provide storage for at least one year of accumulated sediment in the basins following construction and capping of RMU-2.

REFERENCES:

1. Appendix H-7 to the RMU-2 Engineering Report entitled “Final Cover Soil Erosion Estimate” prepared by ARCADIS, August 2009.

2. Figure 1 entitled “Drainage Area Plan,” prepared by ARCADIS, February 2013 (attached).

3. Technical Release 55 “Urban Hydrology for Small Watersheds,” Soil Conservation Service, June 1986.

4. “New York State Standards and Specifications for Erosion and Sediment Control,” New York State Department of Environmental Conservation, April 2005.

5. Site Stormwater Drainage Evaluation, prepared by Blasland, Bouck & Lee, Inc. (now known as ARCADIS), December 2003.

ASSUMPTIONS:

1. The construction and capping of RMU-2 and the construction of related site features (e.g., Fac Ponds 1 and 2, Fac Pond 5, the Drum Management Building, and miscellaneous drainage features and access roads) affects the size and runoff curve number for the watersheds to stormwater retention areas V01, V02, V04, and V05. The tributary watersheds for stormwater retention areas V03 and V06 are unaffected by the construction of RMU-2 and related site features and are therefore not evaluated herein.

2. The required storage volume is based on the need to contain the stormwater runoff from the 25-year, 24-hour storm event (plus one year of sediment accumulation from the RMU-2 watershed).

3. Stormwater runoff volumes determined in this calculation are based on 4.00 inches of rainfall (i.e., the 25-year, 24-hour storm) and the following runoff curve numbers (from Reference 4 assuming a hydrologic group “C):

    - Landfill covers and other mowed areas = 74 (open space, good condition)

Calculation Sheet

- RMU-2 newly graded areas = 91
- Grass areas – not mowed = 71 (meadow)
- Wooded areas = 70 (good condition)
- Impervious areas = 98 (paved roads and building roofs)
- Gravel areas = 89
- Containment areas = 100 (stormwater ponds)

4. The watersheds draining to stormwater retention areas V01, V02, V04, and V05 and the individual acreages for each runoff curve number are based on existing site topography collected for a previous site stormwater drainage evaluation (Reference 5) but modified to account for proposed changes resulting from the RMU-2 project components.

5. To maintain consistency with previous stormwater retention area evaluations for the site, the required storage volume within each retention area is evaluated under two scenarios (interim and final conditions), each with different stormwater runoff conditions, annual sediment accumulations, and freeboard requirements, as outlined below.

   Interim Condition

   The interim condition assumes that approximately half of the RMU-2 tributary area to each basin is newly capped and thus unvegetated. The remainder of the RMU-2 tributary area is assumed to be vegetated. The interim condition is intended to model runoff conditions as establishment of final cover vegetation progresses. Due to the temporary nature of the interim condition, it is assumed that no freeboard requirement exists for this condition.

   Only RMU-2 is considered to be newly capped in this scenario because the other landfills at the site are capped and vegetated at this point and RMU-1 is expected to be fully capped and vegetated prior to the first phase of RMU-2 capping. For the interim condition, sediment accumulation is accounted for from the newly capped portion of RMU-2. Sediment accumulation is not accounted for from other units. An interim condition is not evaluated for stormwater retention area V02 because the basin does not receive any runoff from the RMU-2 final cover.

   Final Condition

   The final condition assumes that the RMU-2 tributary area is completely vegetated. This condition is intended to model runoff conditions following the establishment of final cover vegetation. A minimum of 1 foot of freeboard is required for the final condition. For the final condition, sediment accumulation is not considered to be significant and so is not accounted for from any of the landfill cover systems.

CALCULATIONS:

1. Watershed Delineation and Stormwater Runoff Volumes

Reference 2 presents the watershed delineation for each of the on-site stormwater retention areas following the construction and capping of RMU-2, the reconstruction of Fac Ponds 1 and 2, the construction of Fac Pond 5 and other miscellaneous site features (refer to Attachment 1). As noted in Assumption 1, the watersheds for stormwater retention areas V03 and V06 are unaffected by the construction of the proposed site features. Table 1 summarizes the watershed characteristics for each stormwater retention area and the resulting stormwater runoff volume for the 25-year, 24-hour event.

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Calculation Sheet

Table 1 – Proposed Watersheds and Stormwater Runoff Volumes

Stormwater Retention Area ID	Watershed Condition	Watershed Area [ac]	Composite CN	25-yr, 24-hr Runoff Depth [in]	Runoff Vol. [ac-ft]
V01	Interim	91.51	79	1.96	14.973
	Final		77	1.81	13.818
V02	Final	28.00	78	1.89	4.403
V04	Interim	68.38	83	2.29	13.027
	Final		82	2.20	12.553
V05	Interim	73.72	79	1.96	12.062
	Final		78	1.89	11.592

Spreadsheet output showing the individual curve numbers and acreages and the determination of the composite curve number is included in Attachment 2. A flow schematic illustrating the connectivity between the surface water retention areas and surface water monitoring points is provided for additional information in Attachment 3.

2. Annual Sediment Accumulation from Newly Capped RMU-2 Final Cover Area

As discussed in Assumption 5, the interim condition includes a provision for sediment accumulation from newly constructed RMU-2 final cover areas that are tributary to each stormwater retention area. Based on Reference 1, the annual soil loss rate from the final cover of RMU-2 is approximately 1.7 tons/acre/year under established vegetation conditions. The annual soil loss rates for newly constructed RMU-2 final cover areas is expected to be approximately 100 times greater because the “C” value in the Universal Soil Loss Equation for unvegetated conditions is approximately 100 times that for vegetated conditions. Consequently, the annual soil loss rate from newly constructed RMU-2 final cover areas is estimated to be approximately 170 tons/acre/year. Table 2 presents the RMU-2 final cover acreage draining to each retention area and that is assumed to be newly constructed as well as the calculated estimated sediment accumulation from that acreage. (Stormwater retention area V02 is not included in the table because it does not receive runoff from the RMU-2 final cover.)

Table 2 – Annual Sediment Accumulations from Newly Constructed RMU-2 Final Cover Areas

Stormwater Retention Area ID	Newly Constructed RMU-2 Final Cover Area [ac]	Annual Soil Loss Rate [tons/ac/yr]	Sediment Unit Weight [lbs/ft^3]	Annual Sediment Vol. [ac-ft]
V01	8.48	170	85	0.779
V04	5.97			0.548
V05	4.13			0.379

Notes:

1. Assumed sediment unit weight is based on typical value for silty clay (Reference 4) .

Calculation Sheet

The annual sediment volumes presented above are combined with the calculated 25-year, 24-hour stormwater runoff volumes to determine peak water surface elevations and freeboards for the retention areas under the interim watershed condition.

3. Peak Water Surface Elevations and Minimum Freeboards

Stormwater runoff volumes for the 25-year, 24-hour event are combined with the annual estimated sediment accumulations from RMU-2 for the interim watershed condition to yield a combined required storage volume for V01, V04, and V05. For the final watershed condition, the required storage volume equals the 25-year, 24-hour stormwater runoff volume plus an allowance for 1 foot of freeboard. Table 3 summarizes the combined storage volumes, peak water surface elevations, and resulting minimum freeboards for interim and final conditions. Stormwater retention basin elevation-storage volume data and storage rating curve analyses for V01, V02, V04, and V05 are included in Attachment 4.

Table 3 – Peak Water Surface Elevations and Minimum Freeboards

Stormwater Retention Area ID	Watershed Condition	Runoff Vol. [ac-ft]	Annual Sediment Accumulation Vol. [ac-ft]	Combined Required Storage Vol. [ac-ft]	Peak Water Surface Elevation [fmsl]	Lowest Perimeter El.[2] [fmsl]	Resulting Freeboard [ft]
V01	Interim	14.973	0.779	15.752	315.92	317.50	1.58
	Final	13.818	-[1]	13.818	315.51		1.99
V02	Final	4.403	-[1]	4.403	320.32	321.53	1.21
V04	Interim	13.027	0.548	13.575	313.96	314.78	0.82
	Final	12.553	-[1]	12.553	313.78		1.00
V05	Interim	12.062	0.379	12.441	316.45	317.24	0.79
	Final	11.592	-[1]	11.592	316.24		1.00

Notes:
1. Annual sediment accumulation volumes are not considered for the final watershed condition. (Assumption 5).
2. The lowest perimeter elevations for V01 and V02 are based on existing conditions. The lowest perimeter elevations for V04 and V05 are the perimeter elevations that must be achieved following upgrades to the stormwater retention areas.

The peak water surface elevations and resulting freeboards in the table above are based on the implementation of the following modifications to V04 and V05:

- V04 will need to be upgraded to provide 1 foot of freeboard. It is estimated that approximately 270 cy will need to be filled along the western boundary of V04 for an approximate linear distance of 330 feet to raise the berm elevation from the existing elevation of 313.61 to 314.78.

- V05 will need to be upgraded to provide 1 foot of freeboard. It is estimated that approximately 100 cy will need to be filled along the northern boundary of V05 for an approximate linear distance of 345 feet to raise the existing low point in the berm from elevation 315.84 to 317.24. Alternatively, the existing V05 stormwater retention area could be expanded to the south to provide the additional required capacity.

Figure 1 in Attachment 1 depicts the approximate locations of the recommended modifications to V04 and

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V05. No modifications to V01 nor V02 are necessary.

SUMMARY:

Stormwater retention areas V01 and V02 provide sufficient storage volume based on the assumptions and conditions presented herein. Stormwater retention areas V04 and V05 must be modified in order to provide the design storage volume and the minimum required freeboard of 1 foot.

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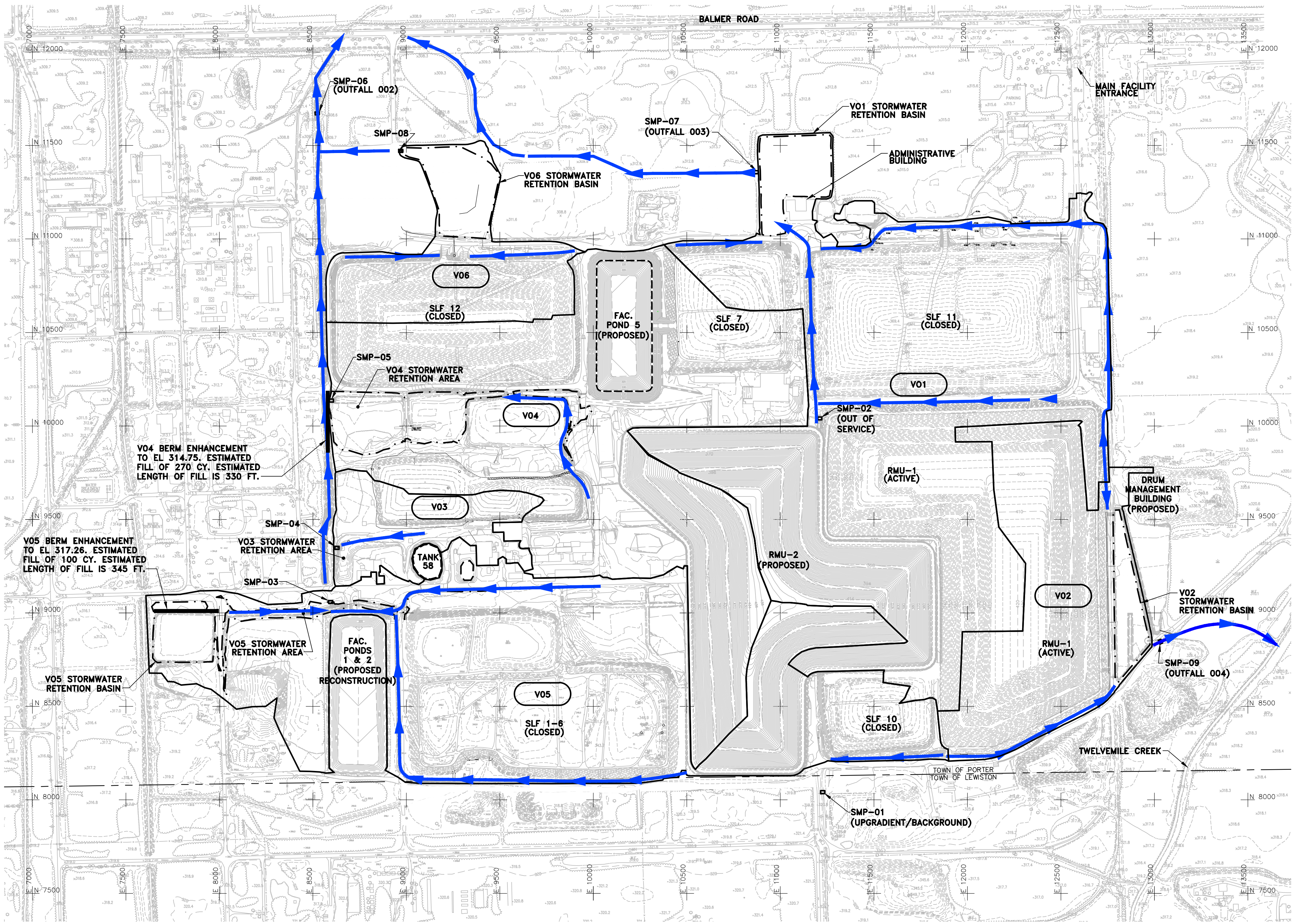
Attachment 1

Watershed Area Map
(Reference 2)



LEGEND:

E 13500 / N 7000	COORDINATE GRID (SEE NOTE 3)
V01	TRIBUTARY DRAINAGE AREA
	DRAINAGE AREA BOUNDARY
	SELF-CONTAINED AREA
	STORMWATER RETENTION BASIN
	EXISTING FLOW DIRECTION
SMP-07	SURFACE WATER MONITORING POINT

DRAINAGE AREA SUMMARY

TRIBUTARY AREA NUMBER	CONTROL GATE	DRAINAGE AREA (ACRES)
V01	SMP-07	91.51
V02	SMP-09	28.00
V03	SMP-04	9.54
V04	SMP-05	68.38
V05	SMP-03	73.72
V06	SMP-08	16.25

GENERAL NOTES:

1. TOPOGRAPHIC BASE MAP INFORMATION INCLUDES: PHOTOGRAMMETRIC METHODS FROM AERIAL PHOTOGRAPHY DATED 5/31/01 BY AIR SURVEY CORP. PROJECT NO. 71010503; RMU-1 PERMITTED FINAL COVER CONTOURS (FINAL COVER IS PARTIALLY CONSTRUCTED); FIELD SURVEYS PERFORMED BY MCINTOSH & MCINTOSH, P.C. DATED 8/00 TO 12/03; FIELD SURVEYS OBTAINED FROM GOLDER ASSOCIATES, INC. DATED 8/01 TO 12/03; FIELD SURVEY OBTAINED FROM URS CORP. DATED 10/03.
2. VERTICAL DATUM BASED ON NGS MEAN SEA LEVEL.
3. GRID COORDINATES SHOWN ARE CWM PLANT GRID.
4. CONTOUR INTERVAL EQUALS 1 FT. AND 2 FT.
5. DASHED CONTOURS INDICATE THAT GROUND IS PARTIALLY OBSCURED BY VEGETATION OR SHADOWS. THESE AREAS MAY NOT MEET STANDARD ACCURACY AND REQUIRE FIELD VERIFICATION.

CWM CHEMICAL SERVICES, LLC
MODEL CITY, NEW YORK
STORMWATER BASIN CAPACITY ANALYSIS

DRAINAGE AREA PLAN

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FIGURE
1

CITY:SYRACUSE DIV/GROUP:141/ENV DB:KMD AGS WLJ PM:(Regd) TM:MBH LYR:ON=*;OFF=*REF*
G:\ENVCAD\SYRACUSE\ACT\B0023725\2011\00002\DWG\SBCA\23725G01.DWG LAYOUT: 1 SAVED: 1/29/2013 10:05 AM ACADVER: 18.1S (LMS TECH) PAGESETUP: ---- PLOTSTYLETABLE: PLTFULL.CTB PLOTTED: 1/29/2013 10:51 AM BY: STEINBERGER, GEORGE
XREFS: IMAGES: PROJECTNAME: ----
23725X01 23725X09 23725X03 23725X06 23725X07 23725X08 23725X02 23725X10 23725X11 23725X12 23725X13 23725X14 23725X05 23725X00 23725X16 23725X18

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Attachment 2

Composite Curve Number
Determination

Proposed Interim Watershed Curve Numbers and Stormwater Runoff Volumes

Drainage Area ID	Watershed Area [acres]	Individual Curve number (CN) Components [acres]							Composite CN	25-yr, 24-hr Runoff Depth [in]	Runoff Volume [acre-feet]
		CN=74 (Landfill covers and other mowed grass)	CN=71 (Unmowed grass)	CN=70 (Woods)	CN=98 (Impervious)	CN=89 (Gravel)	CN=100 (Containment Areas)	CN=91 RMU-2 Area Bare Soil			
V01	91.51	64.69	1.25	2.29	4.69	7.45	2.67	8.48	79	1.96	14.973
V04	68.38	30.64	5.01	0.96	9.10	10.79	5.91	5.97	83	2.29	13.027
V05	73.72	41.26	0.00	12.17	3.91	8.53	3.72	4.13	79	1.96	12.062

Proposed Final Watershed Curve Numbers and Stormwater Runoff Volumes

Drainage Area ID	Watershed Area [acres]	Individual Curve number (CN) Components [acres]						Composite CN	25-yr, 24-hr Runoff Depth [in]	Runoff Volume [acre-feet]
		CN=74 (Landfill covers and other mowed grass)	CN=71 (Unmowed grass)	CN=70 (Woods)	CN=98 (Impervious)	CN=89 (Gravel)	CN=100 (Containment Areas)			
V01	91.51	73.16	1.25	2.29	4.69	7.45	2.67	77	1.81	13.818
V02	28.00	23.01	0.00	0.00	0.75	1.82	2.42	78	1.89	4.403
V04	68.38	36.61	5.01	0.96	9.10	10.79	5.91	82	2.20	12.553
V05	73.72	45.39	0.00	12.17	3.91	8.53	3.72	78	1.89	11.592

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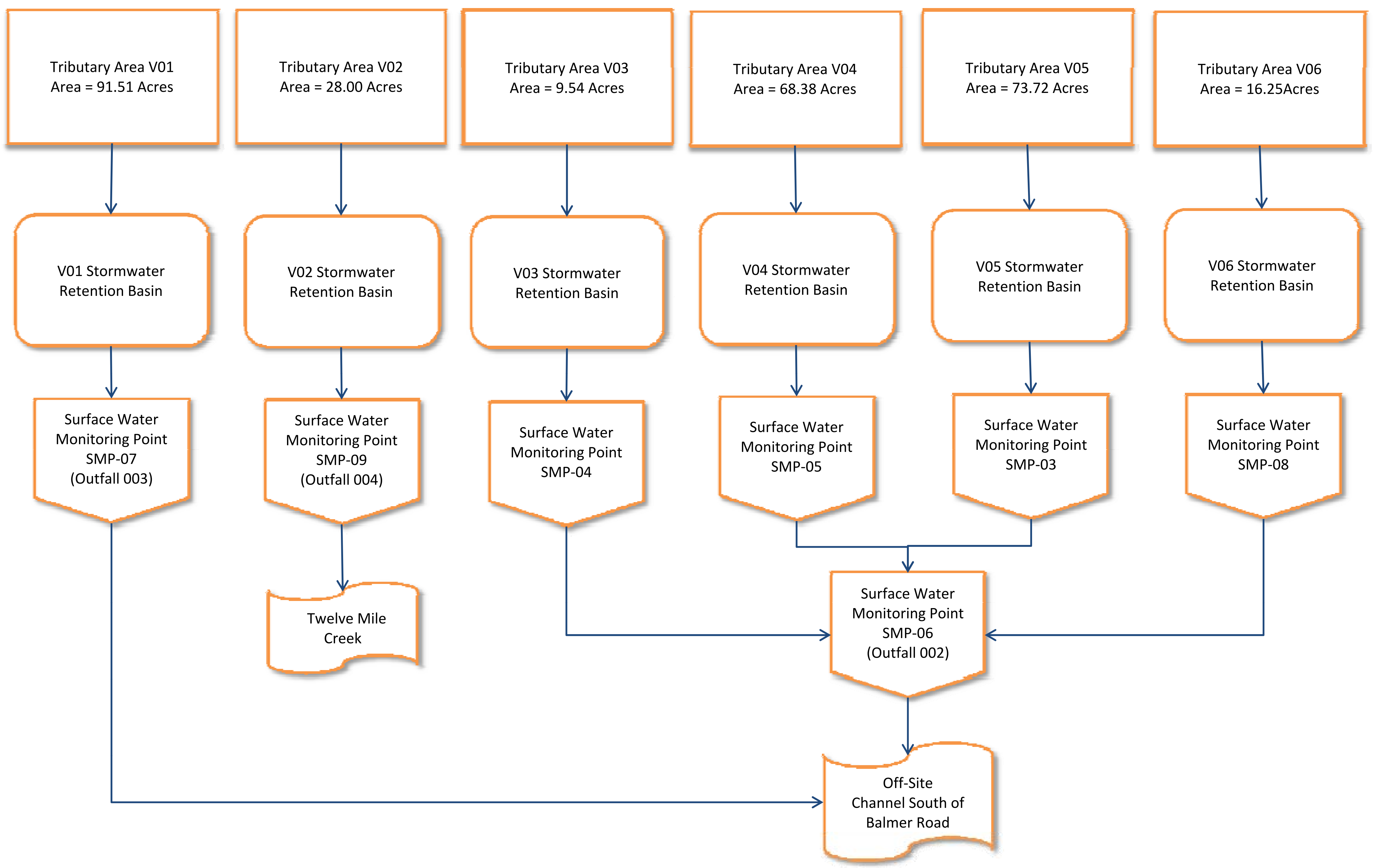
Attachment 3

Surface Water Flow
Schematic

CWM CHEMICAL SERVICES, LLC
MODEL CITY, NEW YORK

SURFACE WATER FLOW SCHEMATIC



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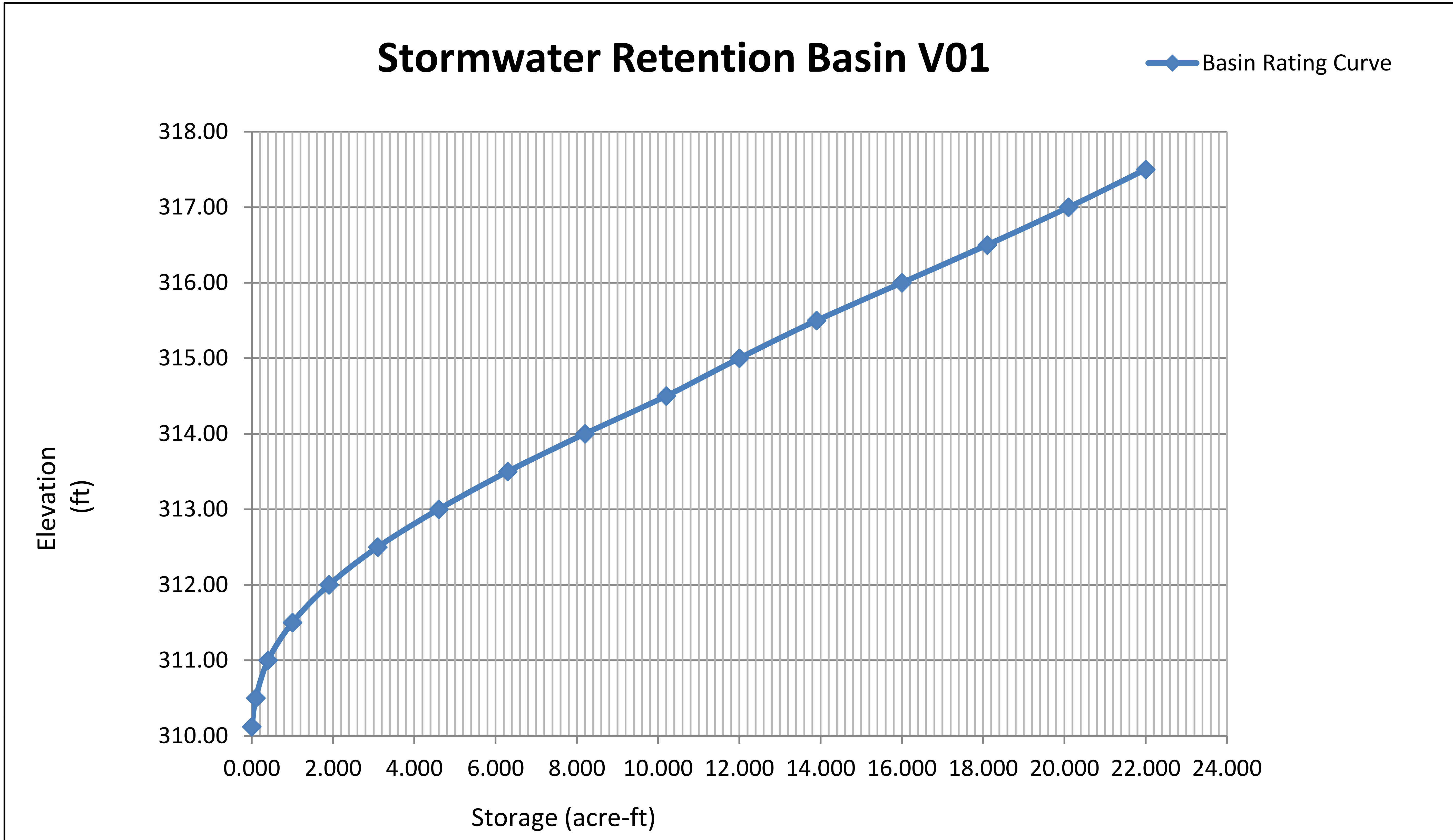
Attachment 4

Stormwater Retention Basin Elevation-Storage Volume Data and Rating Curve Analysis

V01		**V02**		**V04**		**V05**	
Elevation (ft)	Volume (acre-ft)	Elevation (ft)	Volume (acre-ft)	Elevation (ft)	Volume (acre-ft)	Elevation (ft)	Volume (acre-ft)
310.12	0.000	317.62	0.000	306.00	0.000	308.94	0.000
310.50	0.100	318.00	0.089	306.50	0.004	309.00	0.000
311.00	0.400	318.50	0.587	307.00	0.016	309.50	0.001
311.50	1.000	319.00	1.495	307.50	0.054	310.00	0.007
312.00	1.900	319.50	2.526	308.00	0.133	310.50	0.029
312.50	3.100	320.00	3.603	308.50	0.244	311.00	0.085
313.00	4.600	320.50	4.723	309.00	0.378	311.50	0.320
313.50	6.300	321.00	5.887	309.50	0.573	312.00	0.924
314.00	8.200	321.50	7.103	310.00	0.884	312.50	1.779
314.50	10.200			310.50	1.312	313.00	2.781
315.00	12.000			311.00	1.928	313.50	3.865
315.50	13.900			311.50	2.887	314.00	5.006
316.00	16.000			312.00	4.313	314.50	6.204
316.50	18.100			312.50	6.217	315.00	7.476
317.00	20.100			313.00	8.460	315.50	8.888
317.50	22.000			313.50	10.962	315.84	9.979
				313.61	11.588	**316.84**	**14.043**
				314.61	**17.332**		

Note:

1. Bold values represent conditions following retention area modification.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Basin Rating Curve
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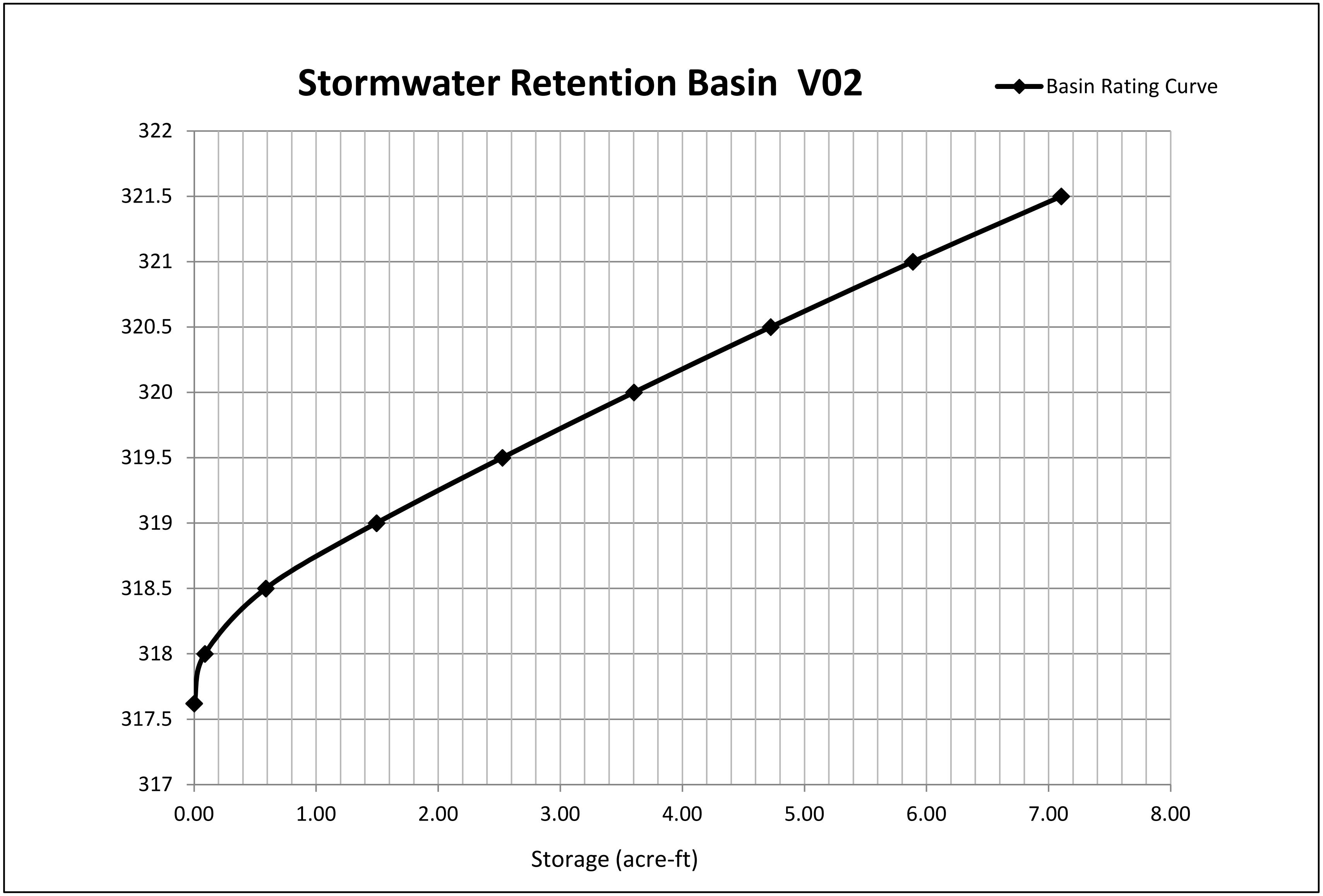
317.00
316.00
315.00
314.00
313.00
312.00
311.00
310.00
Elevation (ft)
0.000
2.000
4.000
6.000
8.000
10.000
12.000
14.000
16.000
18.000
20.000
22.000
24.000
Storage (acre-ft)


Stormwater Retention Basin V02
Basin Rating Curve
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321.5
321
320.5
320
319.5
319
318.5
318
317.5
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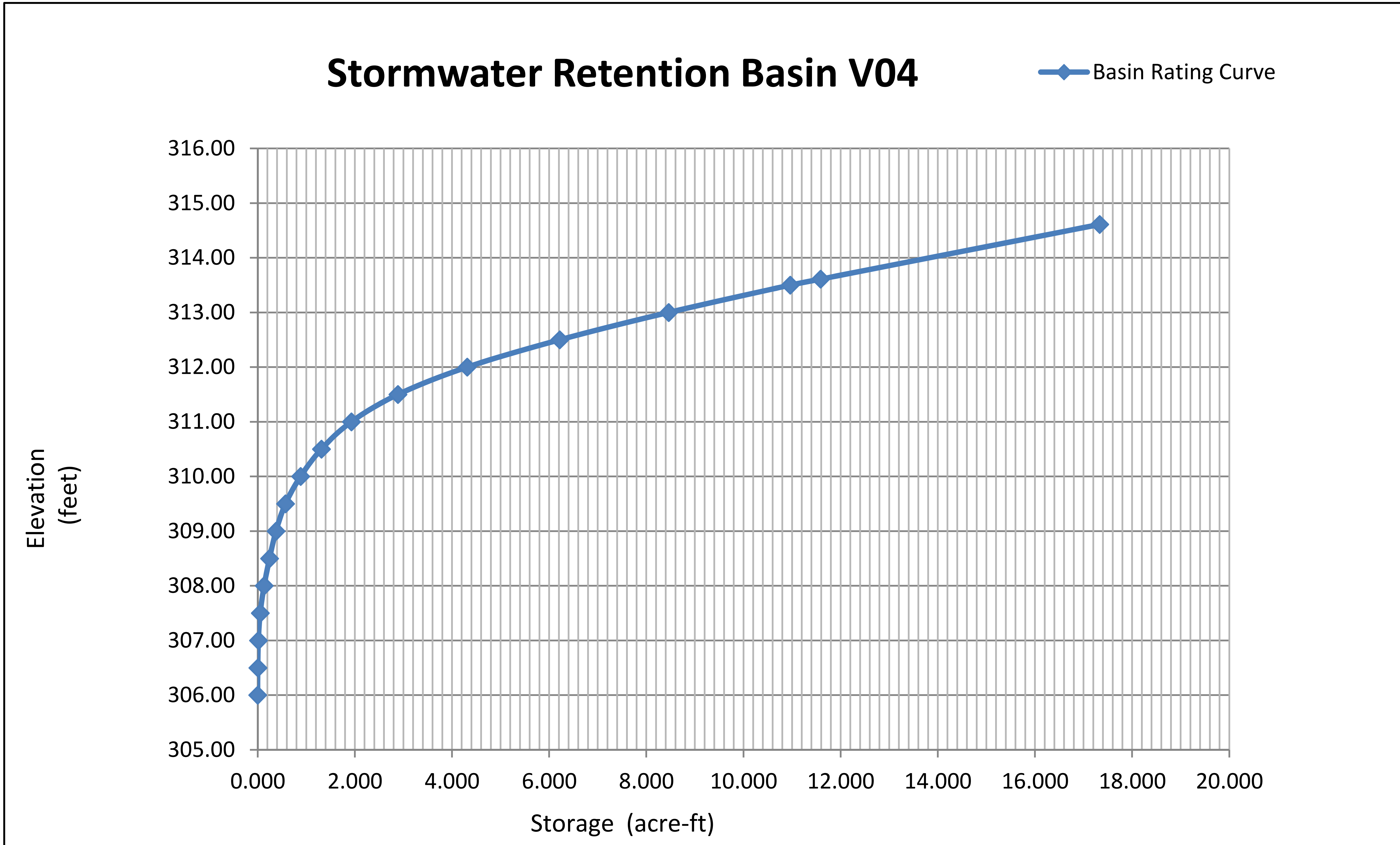
8.00
Storage (acre-ft)


Stormwater Retention Basin V04
Basin Rating Curve
Elevation
(feet)
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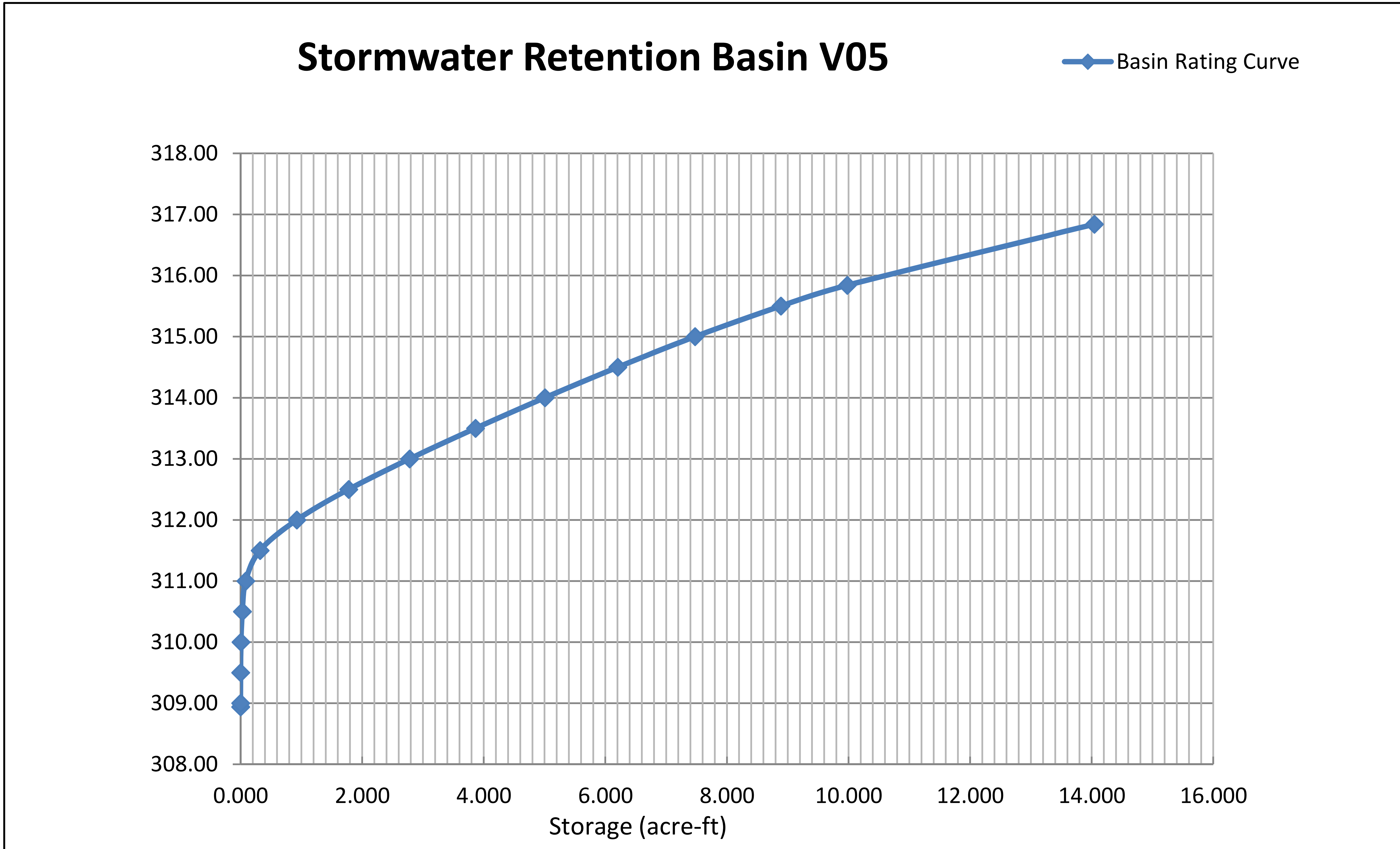
315.00
314.00
313.00
312.00
311.00
310.00
309.00
308.00
307.00
306.00
305.00
0.000
2.000
4.000
6.000
8.000
10.000
12.000
14.000
16.000
18.000
20.000
Storage (acre-ft)


Stormwater Retention Basin V05
Basin Rating Curve
318.00
317.00
316.00
315.00
314.00
313.00
312.00
311.00
310.00
309.00
308.00
0.000
2.000
4.000
6.000
8.000
10.000
12.000
14.000
16.000
Storage (acre-ft)

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Appendix H-9

Fac Pond 5 Channel and Culvert Design

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Calculation Sheet

Client: CWM Chemical Services, LLC

Project Location: Model City, New York

Project: RMU-2 Design Calculations **Project No.: B0023725.2009**

Subject: Appendix H-9: Fac Pond 5 Channel and Culvert Design

Prepared By: PDB **Date: August 2009**

Checked By: BMS **Date: August 2009**

Reviewed By: PHB **Date: August 2009**

OBJECTIVE:

Demonstrate that the proposed cross-sectional geometry of the Fac Pond 5 channel provides adequate hydraulic capacity to convey the estimated peak discharge from the 25-year, 24-hour storm. Demonstrate that stable hydraulic conditions exist in the Fac Pond 5 channel. Determine the required culvert configurations for the Fac Pond 5 channel based on the estimated peak discharge from the 25-year, 24-hour design storm.

REFERENCES:

1. RMU-2 Permit Drawing No. 34 entitled "Fac Pond Grading Plan", ARCADIS, August 2009.
2. Technical Release 55 "Urban Hydrology for Small Watersheds," Soil Conservation Service, June 1986.
3. HydroCAD Software Solutions, LLC, HydroCAD. Version 8.5. Computer Software, 2006. (Output attached).
4. "New York Guidelines for Urban Erosion and Sediment Control," August 2005.
5. Hydraulic Engineering Circular No.15 (HEC15) "Design of Roadside Channels with Flexible Linings", U.S. Department of Transportation Federal Highway Administration, April 1988.
6. Manufacturer Literature Advanced Drainage Systems, Inc.

ASSUMPTIONS:

1. The Fac Pond 5 channel has a trapezoidal geometry with a 2-foot base width and sideslopes of 2H:1V. The channel is located on the eastern and southern edges of Fac Pond 5 and conveys stormwater runoff from portions of SLF 7 and areas north of RMU-2 to the V04 stormwater retention area.
2. The invert slope of the Fac Pond 5 channel is 0.3%.
3. The design storm is the 25-year, 24-hour event, which produces 4.0 inches of rainfall.
4. The runoff curve numbers for the tributary watershed to the Fac Pond 5 channel and culverts include 79 for vegetated areas, 89 for gravel roads, and 98 for buildings and pavement (based on values presented in Reference 2 for Hydrologic Soil Group "C").

Calculation Sheet

5. The Manning “n” value for the Fac Pond 5 channel is based on Reference 4 and assumes riprap with a D_{50} of 3 inches.

6. No minimum freeboard is required for the Fac Pond 5 channel due to the presence of grassed (or otherwise stabilized) conditions adjacent to the edges of the Fac Pond 5 channel (Reference 4).

7. To evaluate the stability of the riprap lining in the channel, a shear stress approach is used assuming a worst-case scenario. This resultant shear stress is based on the peak flowrate for the 25-year, 24-hour design storm.

8. The Fac Pond 5 channel utilizes two culverts. The first culvert is used to convey channel flow under the Fac Pond 5 access ramp located on the eastern edge of Fac Pond 5. The second culvert is used to convey channel flow under the stabilization facility access road and into the V04 stormwater retention area. Both culverts use identical pipe materials and are designed using the same methodology.

9. The required culvert configurations are based on the estimated peak discharge from the 25-year, 24-hour storm event. Each culvert configuration is deemed acceptable if the design can convey the 25-year, 24-hour estimated peak discharge without causing a headwater depth that exceeds the depth of the channel in which the culverts are installed. The Fac Pond 5 channel depth varies depending on location. Fac Pond 5 channel depths at the culvert locations are indicated herein.

10. The flow capacity of each culvert is modeled using Reference 3, which accounts for both pipe friction losses and energy losses at the culvert entrance and exit. The model also considers dynamic tailwater conditions due to downstream culverts where applicable.

11. All culvert pipes are smooth-bore corrugated HDPE pipes having Manning “n” value of 0.012 based on Reference 6. The inlet and outlet of each culvert pipe are mitered to conform to the slope of the ditch to reduce entrance and exit energy losses. The same inlet and outlet condition can be obtained with a flared-end section.

12. Culvert pipes are sloped at 0.3%.

CALCULATIONS:

1. Estimated Peak Discharges

The tributary watersheds that are used to design and evaluate the Fac Pond 5 channel and culverts are depicted on a watershed map included in Attachment 1. Table 1 summarizes the runoff characteristics for the tributary watersheds draining to the channel and culverts and the 25-year, 24-hour estimated peak discharges.

Table 1 – Watershed Characteristics

Watershed ID	Watershed Area [acres]	Runoff Curve Number	Time of Concentration [min]	25-yr, 24-hr Estimated Peak Discharge [cfs]
Area 1	2.64	80	10.0	8.2
Area 2	10.71	85	25.2	25.2

Supporting output from HydroCAD is included in Attachment 2.

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Calculation Sheet

2. Resulting Hydraulic Conditions

Based on the above estimated peak discharges and Reference 3, Tables 2 and 3 summarize the resulting hydraulic conditions at the culvert inverts and within the Fac Pond 5 channel, respectively.

Table 2 – Flow Characteristics at Culvert Inlets

Channel ID	Culvert Diameter [in]	Contributing Watersheds	Resulting Peak Discharge [cfs]	Manning "n"	Channel Depth[1] [in]	Flow Depth at Culvert Inlet [in]
Fac Pond 5 Ramp Culvert	24	Area 1	8.2	0.012	28	19
Stabilization Facility Road Culvert	36	Area 1 & Area 2	28.6	0.012	50	33

Notes:
1.) Channel depth represents the depth of the channel at the location of the culvert.

As shown in Table 2 the Fac Pond 5 culverts provide sufficient capacity to convey the estimated peak flowrate from the 25-year, 24-hour design storm. Supporting output is included in Attachment 2.

Table 3 – Flow Characteristics in Channel

Channel ID	Contributing watersheds	25-yr, 24-hr Estimated Peak Discharge [cfs]	Manning "n"	Channel Depth [in]	Flow Depth [in]	Flow Velocity [cfs]
Fac Pond 5 Channel	Area 1 & Area 2	28.6	0.033	24	23	2.56

A minimum channel depth of 24 inches for the Fac Pond 5 channel provides adequate hydraulic capacity to convey the 25-year, 24-hour estimated peak discharge. Supporting output is included in Attachment 2.

2. Shear Stress Analysis for Riprap Used to Line Channels

To calculate the maximum shear stress, τ_b, on the bed of a channel using Reference 5:

$$\tau_b(\frac{lb}{ft^2}) = (\gamma_w)(y)(i)$$

where
γ_w = Unit weight of water (62.4 lb/ft^3)
y = Depth of flow (ft) = 1.91 ft
i = Bed slope (ft/ft)= 0.003 ft/ft

To calculate the allowable shear stress, τ_c, on the bed of the channel using Reference 5:

$$\tau_c(\frac{lb}{ft^2}) = 4D_{50} \ ,$$

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Calculation Sheet

where
D50 = 0.25 ft (Assumption 5)

$$\tau_c = 1.0 \frac{lb}{ft^2}$$

The factor of safety is then determined by

Factor of Safety = $\dfrac{Allowable}{Maximum} = \dfrac{\tau_c}{\tau_b}$

To calculate the maximum shear stress, τ_m, on the sideslope of a channel using Reference 5:

$$\tau_m\left(\frac{lb}{ft^2}\right) = .75(\gamma_w)(y)(i)$$

where,
γ_w = Unit weight of water (62.4 lb/ft^3)
y = Depth of flow (ft)= 1.91
i = Bed slope (ft/ft)= .003 ft/ft

To calculate the allowable shear stress, τ_s, on the sideslope of the channel using Reference 5:

$$\tau_s\left(\frac{lb}{ft^2}\right) = (\tau_c)\left(\sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}}\right)$$

θ = side angle of the channel = tan $^{-1}$ (1/2) = 26.6 degrees
ϕ = angle of repose = 40 degrees

$$\tau_s = 0.72 \frac{lb}{ft^2}$$

The shear stress analysis for the sideslopes of the channel is summarized in Table 4 below.

Table 4 - Flow Characteristics and Resulting Shear Stresses on Channel Bed and Sideslopes

Channel Location	Flowrate (cfs)	Flow Depth (ft)	Resulting Shear Stress (lb/ft^2)	Allowable Shear Stress (lb/ft^2)	Factor of Safety
Channel Bed	28.6	1.91	0.36	1.00	2.79
Channel Sideslopes	28.6	1.91	0.27	0.72	2.67

Calculation Sheet

As indicated in Table 4, the Fac Pond 5 channel is hydraulically stable. Supporting output is included in Attachment 3.

SUMMARY:

Based on the above calculations, the Fac Pond 5 culverts provide sufficient capacity to convey the estimated peak flowrate from the 25-year, 24-hour design storm. The proposed cross-sectional geometry of the Fac Pond 5 channel provides adequate hydraulic capacity to convey the estimated peak discharge from the 25-year, 24-hour storm. Conditions in the Fac Pond 5 channel are hydraulically stable.

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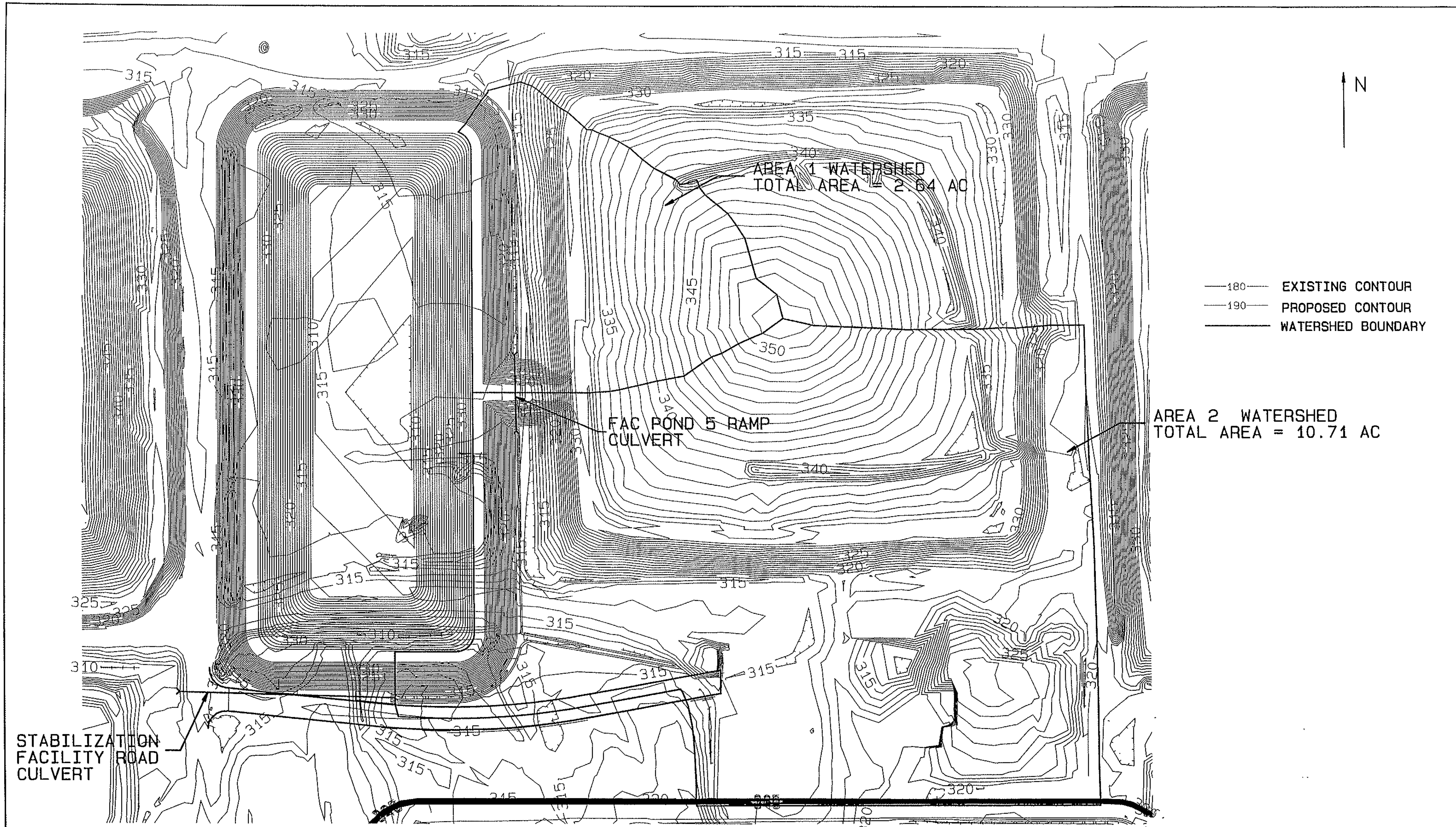
Attachment 1

Watershed Map



SCALE IN FEET
0 120 240

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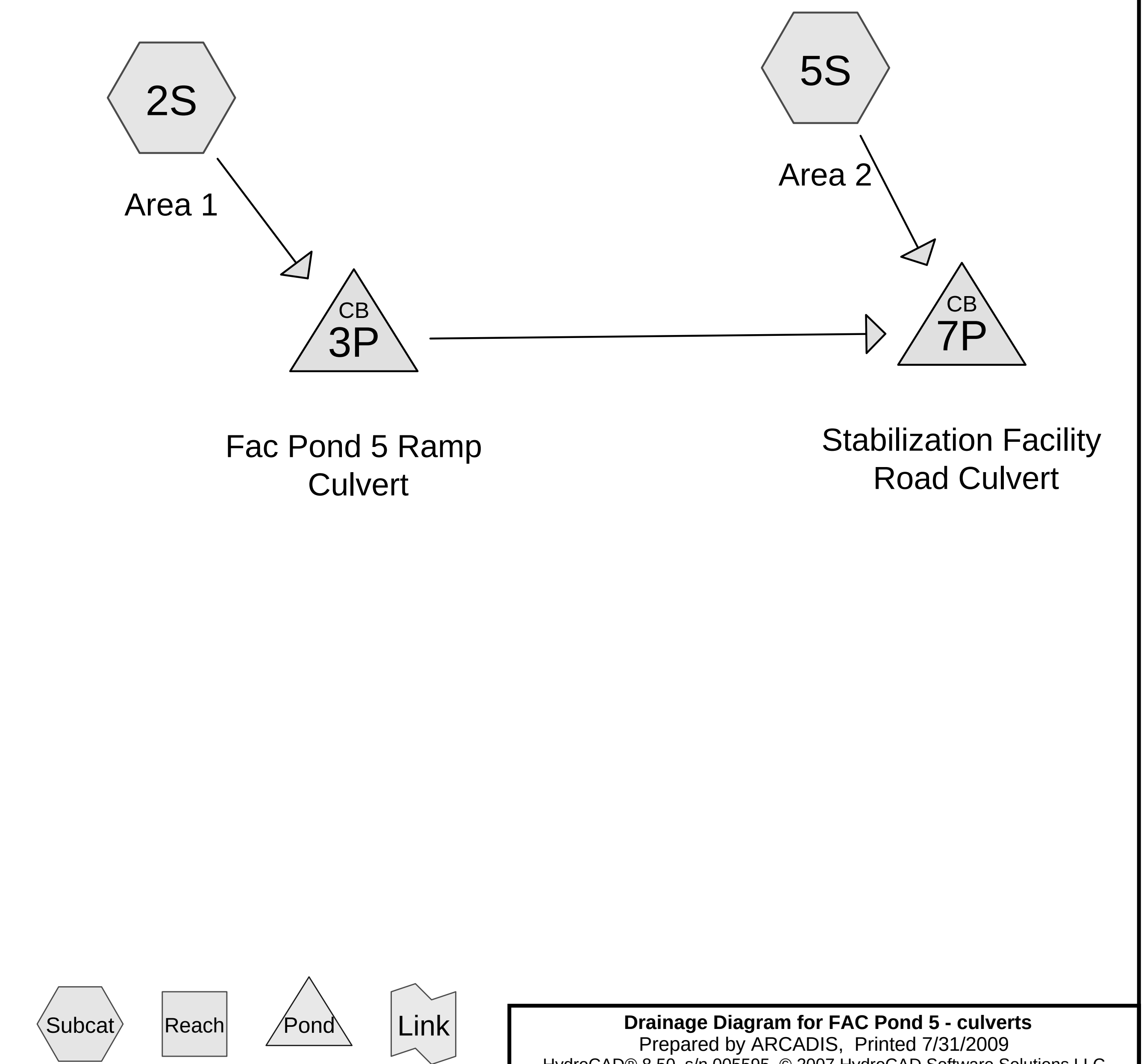
SHEET TITLE: SLF 7 DIVERSION CHANNEL WATERSHED
PROJECT TITLE: RMU-2 PERMIT CALCULATION
CWM CHEMICAL SERVICES
PROJECT LOCATION: MODEL CITY, NEW YORK
PROJECT NO: B0023725.2009
DATE: AUGUST 2009 BY: PDB FILE NO.: 23725.PRO

Figure 1

ARCADIS

Attachment 2

HydroCAD Output



Drainage Diagram for FAC Pond 5 - culverts
Prepared by ARCADIS, Printed 7/31/2009
HydroCAD® 8.50 s/n 005595 © 2007 HydroCAD Software Solutions LLC

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
9.530	79	50-75% Grass cover, Fair, HSG C (2S,5S)
1.350	89	Gravel roads, HSG C (2S,5S)
2.470	98	Paved parking & roofs (5S)
13.350		**TOTAL AREA**

Time span=1.00-30.00 hrs, dt=0.05 hrs, 581 points
Runoff by SCS TR-20 method, UH=SCS
Reach routing by Dyn-Stor-Ind method - Pond routing by Dyn-Stor-Ind method

Subcatchment 2S: Area 1 Runoff Area=2.640 ac 0.00% Impervious Runoff Depth=2.04"
Flow Length=753' Tc=10.0 min CN=80 Runoff=8.16 cfs 0.449 af

Subcatchment 5S: Area 2 Runoff Area=10.710 ac 23.06% Impervious Runoff Depth=2.46"
Flow Length=1,713' Tc=25.2 min CN=85 Runoff=25.42 cfs 2.194 af

Pond 3P: Fac Pond 5 Ramp Culvert Peak Elev=314.87' Inflow=8.16 cfs 0.449 af
24.0" x 101.0' Culvert Outflow=8.16 cfs 0.449 af

Pond 7P: Stabilization Facility Road Culvert Peak Elev=314.21' Inflow=28.64 cfs 2.643 af
36.0" x 280.0' Culvert Outflow=28.64 cfs 2.643 af

Total Runoff Area = 13.350 ac Runoff Volume = 2.643 af Average Runoff Depth = 2.38"
81.50% Pervious = 10.880 ac 18.50% Impervious = 2.470 ac

Summary for Subcatchment 2S: Area 1

Runoff = 8.16 cfs @ 12.02 hrs, Volume= 0.449 af, Depth= 2.04"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 1.00-30.00 hrs, dt= 0.05 hrs
Type II 24-hr Rainfall=4.00"

Area (ac)	CN	Description
2.490	79	50-75% Grass cover, Fair, HSG C
0.150	89	Gravel roads, HSG C
2.640	80	Weighted Average
2.640		Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.1	75	0.0500	0.20		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.50"
1.5	396	0.0700	4.26		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
2.4	282	0.0030	1.93	33.85	**Channel Flow,** Area= 17.5 sf Perim= 25.2' r= 0.69' n= 0.033
10.0	753	Total			

Summary for Subcatchment 5S: Area 2

Runoff = 25.42 cfs @ 12.19 hrs, Volume= 2.194 af, Depth= 2.46"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 1.00-30.00 hrs, dt= 0.05 hrs
Type II 24-hr Rainfall=4.00"

Area (ac)	CN	Description
7.040	79	50-75% Grass cover, Fair, HSG C
2.470	98	Paved parking & roofs
1.200	89	Gravel roads, HSG C
10.710	85	Weighted Average
8.240		Pervious Area
2.470		Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
7.5	75	0.0300	0.17		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.50"
1.3	342	0.0700	4.26		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
12.6	860	0.0050	1.14		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
3.8	436	0.0030	1.93	33.85	**Channel Flow,** Area= 17.5 sf Perim= 25.2' r= 0.69' n= 0.033
25.2	1,713	Total			

Summary for Pond 3P: Fac Pond 5 Ramp Culvert

Inflow Area =	2.640 ac,	0.00% Impervious, Inflow Depth = 2.04"	
Inflow =	8.16 cfs @	12.02 hrs, Volume=	0.449 af
Outflow =	8.16 cfs @	12.02 hrs, Volume=	0.449 af, Atten= 0%, Lag= 0.0 min
Primary =	8.16 cfs @	12.02 hrs, Volume=	0.449 af

Routing by Dyn-Stor-Ind method, Time Span= 1.00-30.00 hrs, dt= 0.05 hrs
Peak Elev= 314.87' @ 12.02 hrs

Device	Routing	Invert	Outlet Devices
#1	Primary	313.27'	**24.0" x 101.0' long Culvert** CPP, mitered to conform to fill, Ke= 0.700 Outlet Invert= 312.97' S= 0.0030 '/' Cc= 0.900 n= 0.012

Primary OutFlow Max=7.95 cfs @ 12.02 hrs HW=314.85' TW=313.89' (Dynamic Tailwater)
└─**1=Culvert** (Barrel Controls 7.95 cfs @ 4.11 fps)

Summary for Pond 7P: Stabilization Facility Road Culvert

Inflow Area = 13.350 ac, 18.50% Impervious, Inflow Depth = 2.38"
Inflow = 28.64 cfs @ 12.14 hrs, Volume= 2.643 af
Outflow = 28.64 cfs @ 12.14 hrs, Volume= 2.643 af, Atten= 0%, Lag= 0.0 min
Primary = 28.64 cfs @ 12.14 hrs, Volume= 2.643 af

Routing by Dyn-Stor-Ind method, Time Span= 1.00-30.00 hrs, dt= 0.05 hrs
Peak Elev= 314.21' @ 12.14 hrs

Device	Routing	Invert	Outlet Devices
#1	Primary	311.50'	**36.0" x 280.0' long Culvert** CPP, mitered to conform to fill, Ke= 0.700 Outlet Invert= 310.66' S= 0.0030 '/' Cc= 0.900 n= 0.012

Primary OutFlow Max=28.56 cfs @ 12.14 hrs HW=314.21' (Free Discharge)
└─**1=Culvert** (Barrel Controls 28.56 cfs @ 5.61 fps)

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Attachment 3

Hydraulic Analysis

Project: RMU-2 Design By: PDB Date: August 2009
Project No.: B0023725.2009
Subject: FAC Pond 5 Channel and Culvert Design Ckd By: BMS Date: August 2009

Riprap Lining

Channel Design (Input)	
Flow Capacity (cfs)	28.60
Base Width (ft)	2.00
Left Side Slope (x:1)	2.00
Right Side Slope (x:1)	2.00
Bed Slope	0.003
Minimum Riprap D_{50}, (in.)	3.0
Maximum Allowable Shear Stress on Bed (psf)	1.00
Maximum Allowable Shear Stress on Sideslopes (psf)	0.72
Manning "n"	0.033

Flow Conditions (Output)	
Flowrate from Manning Equation (cfs)	28.60
Required Flow Depth (ft)	1.91
Resulting Flow Velocity (ft/s)	2.56
Resulting Flow Width at Top (ft)	9.65
Resulting Flow Area (ft^2)	11.15
Resulting Wetted Perimeter (ft)	10.56
Resulting Hydraulic Radius (ft)	1.06
Resulting Shear Stress on Bed (psf)	0.36
Resulting Shear Stress on Sideslopes (psf)	0.27
Channel Dimensions	
Channel Depth (ft)	0.75
Resulting Freeboard (ft)	-1.16
Shear Stress Factor of Safety (Bed)	2.79
Shear Stress Factor of Safety (Sideslope)	2.67

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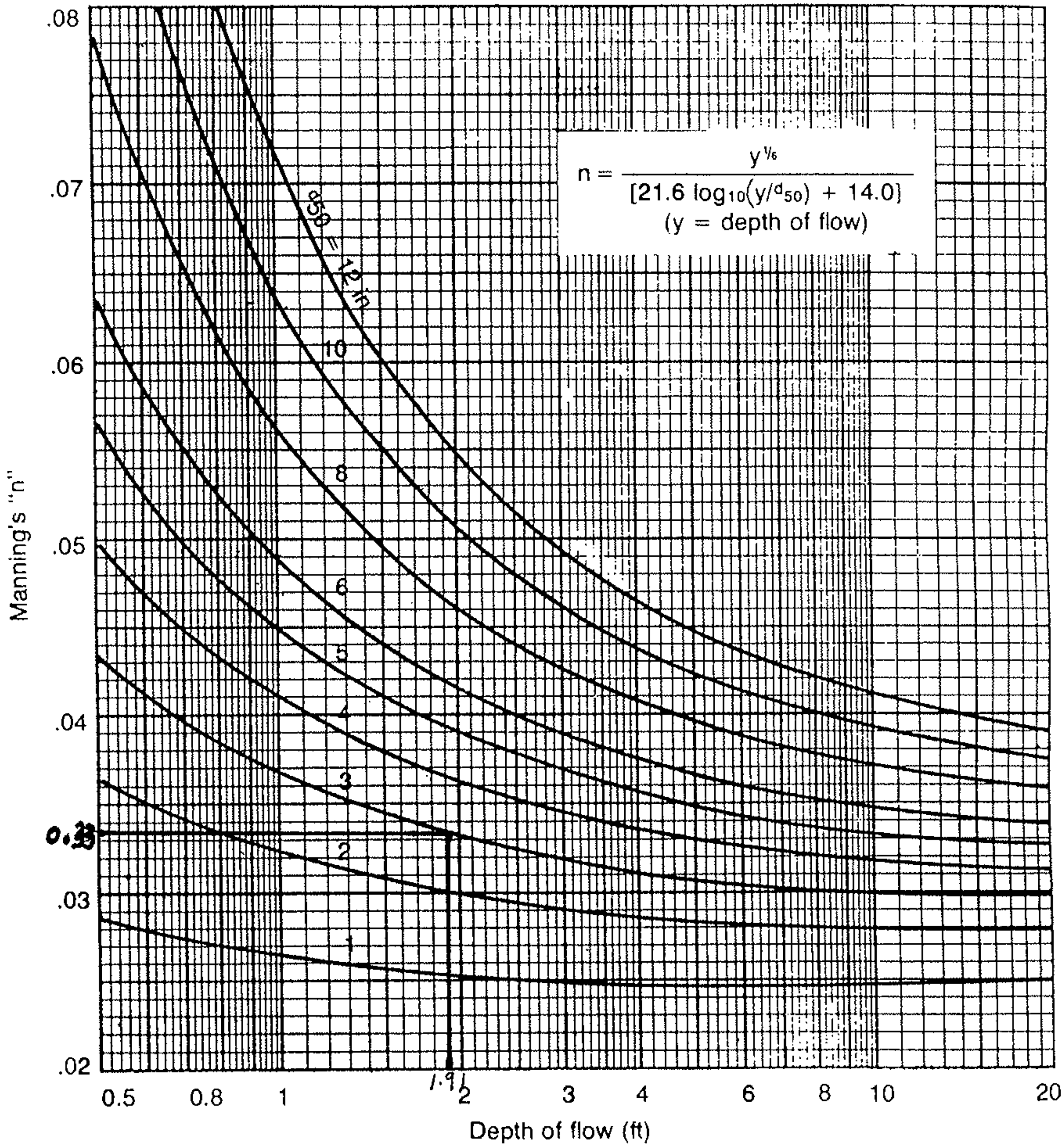
Attachment 4

References

Figure 5B.11
Determining "n" for Riprap Lined Channel using Depth of Flow
(USDA - NRCS)



Appendix H-10

SLF 10 Ditch Design

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Imagine the result

Calculation Sheet

Client: CWM Chemical Services, LLC

Project Location: Model City, New York

Project: RMU-2 Design Calculations **Project No.: B0023725.2009**

Subject: Appendix H-10: SLF 10 Ditch

Prepared By: GNG **Date: August 2009**

Checked By: BMS **Date: August 2009**

Reviewed By: PHB **Date: August 2009**

OBJECTIVE:

Demonstrate that the proposed cross-sectional geometry of the SLF 10 ditch provides adequate hydraulic capacity to convey the estimated peak discharge from the 25-year, 24-hour storm. Demonstrate that stable hydraulic conditions exist in the SLF 10 ditch.

REFERENCES:

1. RMU-2 Permit Drawing No. 7 entitled “Top of Vegetative Cover Grades”, ARCADIS, August 2009.

2. Technical Release 55 “Urban Hydrology for Small Watersheds,” Soil Conservation Service, June 1986.

3. HydroCAD Software Solutions, LLC, HydroCAD. Version 8.5. Computer Software, 2006. (Output attached).

4. “New York Guidelines for Urban Erosion and Sediment Control,” August 2005.

5. Hydraulic Engineering Circular No.15 (HEC-15) “Design of Roadside Channels with Flexible Linings”, U.S. Department of Transportation Federal Highway Administration, April 1988.

ASSUMPTIONS:

1. The SLF 10 ditch is located on the eastern edge of RMU-2 adjacent to Cell 19 and conveys stormwater runoff from portions of SLF 10 to the RMU-2 south ditch and ultimately to the V05 retention basin (Reference 1). The SLF 10 ditch has a trapezoidal geometry with a 2-foot base width and sideslopes of 2H:1V on the eastern edge and 0.25H:1V (MSE wall slope) on the western edge. The minimum channel depth is 2-feet.

2. The invert slope of the SLF 10 ditch is 0.3%.

3. The design storm is the 25-year, 24-hour event, which produces 4.0 inches of rainfall.

4. Runoff curve numbers for the tributary watershed to the SLF 10 ditch include 79 for capped areas with established vegetation and 89 for gravel roads and riprap-lined ditches (based on Reference 2 for Hydrologic Soil Group “C”).

5. The Manning “n” value for the SLF 10 ditch is based on Reference 4 and assumes riprap with a D_{50} of 3 inches.

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Calculation Sheet

6. No minimum freeboard is required for the ditch due to the presence of grass (or otherwise stabilized) conditions adjacent to the edges of the SLF 10 ditch (Reference 4).

7. To evaluate the stability of the riprap lining in the ditch, a shear stress approach is used assuming a worst-case scenario. The resultant shear stress is based on the peak flowrate for the 25-year, 24-hour design storm.

CALCULATIONS:

1. Estimated Peak Discharges

The tributary watershed that is used to design and evaluate the SLF 10 ditch is depicted on the watershed map included in Attachment 1. Table 1 summarizes the runoff characteristics for the tributary watershed draining to the ditch and the 25-year, 24-hour estimated peak discharge.

Table 1 – Watershed Characteristics

Watershed Area [acres]	Runoff Curve Number	Time of Concentration [min]	25-yr, 24-hr Estimated Peak Discharge [cfs]
5.12	80	34.5	8.1

Supporting output from HydroCAD is included in Attachment 2.

2. Resulting Hydraulic Conditions

Table 2 summarizes the resulting hydraulic conditions within the SLF 10 ditch.

Table 2 – Flow Characteristics

25-yr, 24-hr Estimated Peak Discharge [cfs]	Manning "n"	Minimum Channel Depth [in]	Flow Depth [in]	Flow Velocity [ft/s]
8.1	0.035	24	15	1.9

A minimum channel depth of 24 inches for the SLF 10 ditch provides adequate hydraulic capacity to convey the 25-year, 24-hour estimated peak discharge. Supporting output is included in Attachment 3.

3. Shear Stress Analysis for Riprap Ditch Lining

To calculate the maximum shear stress, τ_b, on the bed of a channel using Reference 5:

$$\tau_b\left(\frac{lb}{ft^2}\right) = (\gamma_w)(y)(i)$$

Where,

γ_w = Unit weight of water (62.4 lb/ft^3)
y = Depth of flow (ft) = 1.27 ft
i = Bed slope (ft/ft)= 0.003 ft/ft

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Calculation Sheet

To calculate the allowable shear stress, τ_c , on the bed of the channel using Reference 5:

$$\tau_c (\frac{lb}{ft^2}) = 4D_{50}$$

Where,
D50 = 0.25 ft (Assumption 5)

The factor of safety is then determined by

Factor of Safety = $\frac{Allowable}{Maximum} = \frac{\tau_c}{\tau_b}$

To calculate the maximum shear stress, τ_m , on the sideslope of a channel using Reference 5:

$$\tau_m (\frac{lb}{ft^2}) = .75(\gamma_w)(y)(i)$$

Where,
γ_w = Unit weight of water (62.4 lb/ft^3)
y = Depth of flow (ft) = 1.27
i = Bed slope (ft/ft)= .003 ft/ft

To calculate the allowable shear stress, τ_s , on the sideslope of the channel using Reference 5:

$$\tau_s (\frac{lb}{ft^2}) = (\tau_c)\left(\sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}}\right)$$

θ = side angle of the channel = tan $^{-1}$ (1/2) = 26.6 degrees
ϕ = angle of repose of rip rap = 40 degrees

The shear stress analysis for the bed and sideslopes of the channel is summarized in Table 3 below.

Table 3 - Shear Stresses on Bed and Sideslopes

Channel Surface	Flowrate (cfs)	Flow Depth (ft)	Resulting Shear Stress (lb/ft^2)	Allowable Shear Stress (lb/ft^2)	Factor of Safety
Bed	8.1	1.27	0.24	1.00	4.21
Sideslopes	8.1	1.27	0.18	0.38	2.12

As indicated in Table 3, the SLF 10 ditch is hydraulically stable. Supporting output is included in Attachment 3.

Infrastructure, environment, buildings

Imagine the result

Calculation Sheet

SUMMARY:

Based on the above calculations, the SLF 10 ditch provides sufficient capacity to convey the estimated peak flowrate from the 25-year, 24-hour design storm. The riprap lining in the SLF 10 ditch is hydraulically stable while conveying the peak flowrate from the 25-year, 24-hour design strom.

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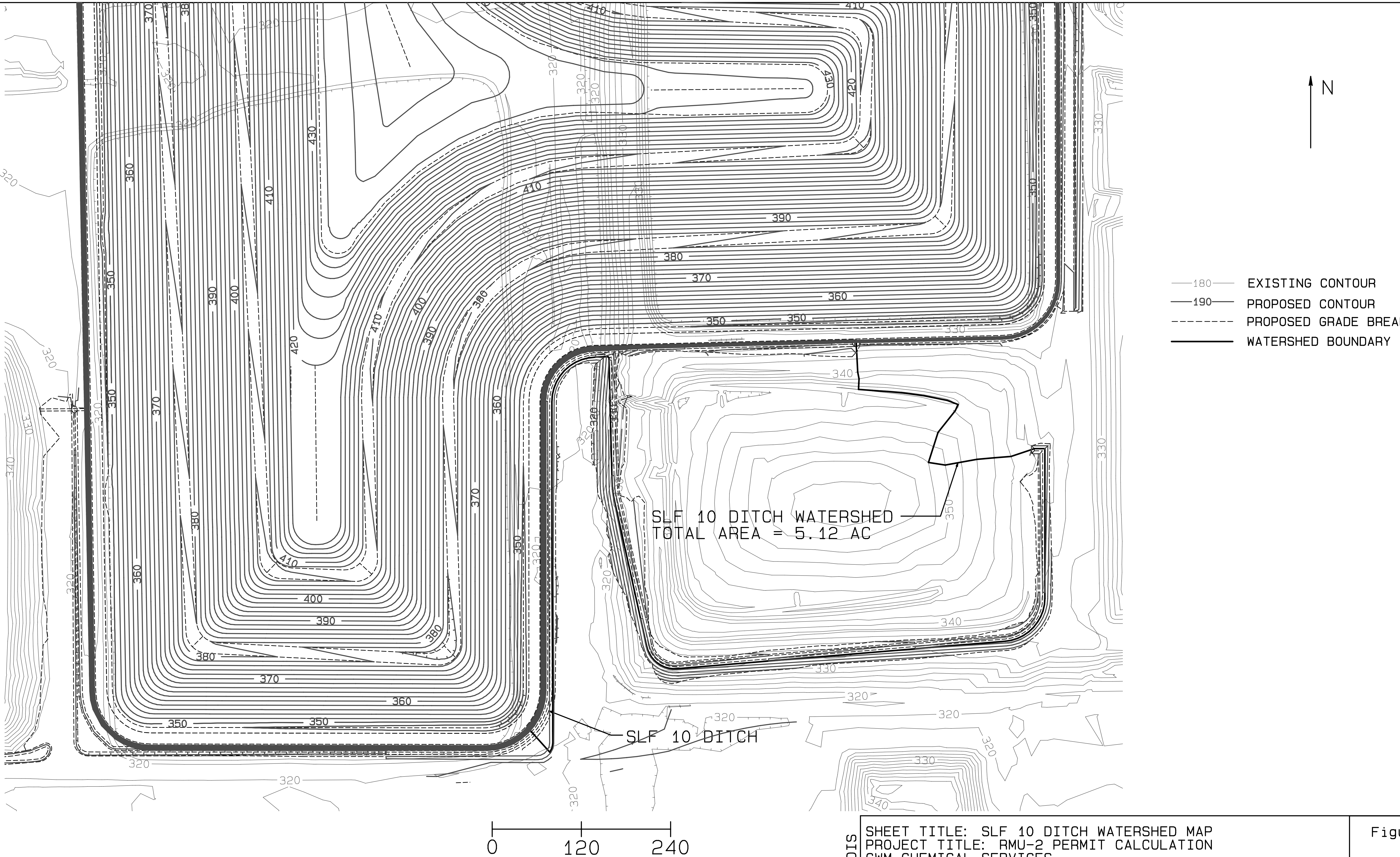
Attachment 1

Watershed Map



ARCADIS		
SHEET TITLE: SLF 10 DITCH WATERSHED MAP PROJECT TITLE: RMU-2 PERMIT CALCULATION CWM CHEMICAL SERVICES PROJECT LOCATION: MODEL CITY, NEW YORK PROJECT NO: B0023725.2009 DATE: AUGUST 2009 BY: GNG	FILE NO.: 23725.PRO	Figure 1

ARCADIS

Attachment 2

HydroCAD Output

Summary for Subcatchment 1S: SLF-10 Ditch

Runoff = 8.06 cfs @ 12.31 hrs, Volume= 0.871 af, Depth= 2.04"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type II 24-hr 25-year Rainfall=4.00"

	Area (ac)	CN	Description
	4.520	79	50-75% Grass cover, Fair, HSG C
	0.340	89	Gravel roads, HSG C
*	0.260	89	Gravel, HSG C
	5.120	80	Weighted Average
	5.120		Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.7	75	0.0600	0.22		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.50"
0.5	152	0.1000	5.09		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
24.0	1,130	0.0100	0.78	15.20	**Channel Flow, SLF 10 DIVERSION CHAN** Area= 19.4 sf Perim= 32.3' r= 0.60' n= 0.135
5.2	575		1.84		**Direct Entry, SLF 10 DITCH**
35.4	1,932	Total			

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Attachment 3

Hydraulic Analysis

Project: <u>RMU-2 Design</u>
Project No.: <u>B0023725.2009</u>
Subject: <u>SLF 10 Ditch</u>

Prepared By: <u>GNG</u>
Date: <u>August 2009</u>
Checkd By:____
Date:____

Riprap Lining

Channel Design (Input)	
Flow Capacity (cfs)	8.06
Base Width (ft)	2.00
Left Side Slope (x:1)	2.00
Right Side Slope (x:1)	0.25
Bed Slope	0.003
Minimum Riprap D50, (in.)	3.0
Maximum Allowable Shear Stress on Bed (psf)	1.00
Maximum Allowable Shear Stress on Sideslopes (psf)	0.38
Manning "n"	0.035

Flow Conditions (Output)	
Flowrate from Manning Equation (cfs)	8.06
Required Flow Depth (ft)	1.27
Resulting Flow Velocity (ft/s)	1.85
Resulting Flow Width at Top (ft)	4.86
Resulting Flow Area (ft^2)	4.35
Resulting Wetted Perimeter (ft)	6.15
Resulting Hydraulic Radius (ft)	0.71
Resulting Shear Stress on Bed (psf)	0.24
Resulting Shear Stress on Sideslopes (psf)	0.18
Channel Dimensions	
Channel Depth (ft)	2.00
Resulting Freeboard (ft)	0.73
Shear Stress Factor of Safety (Bed)	4.21
Shear Stress Factor of Safety (Sideslope)	2.12

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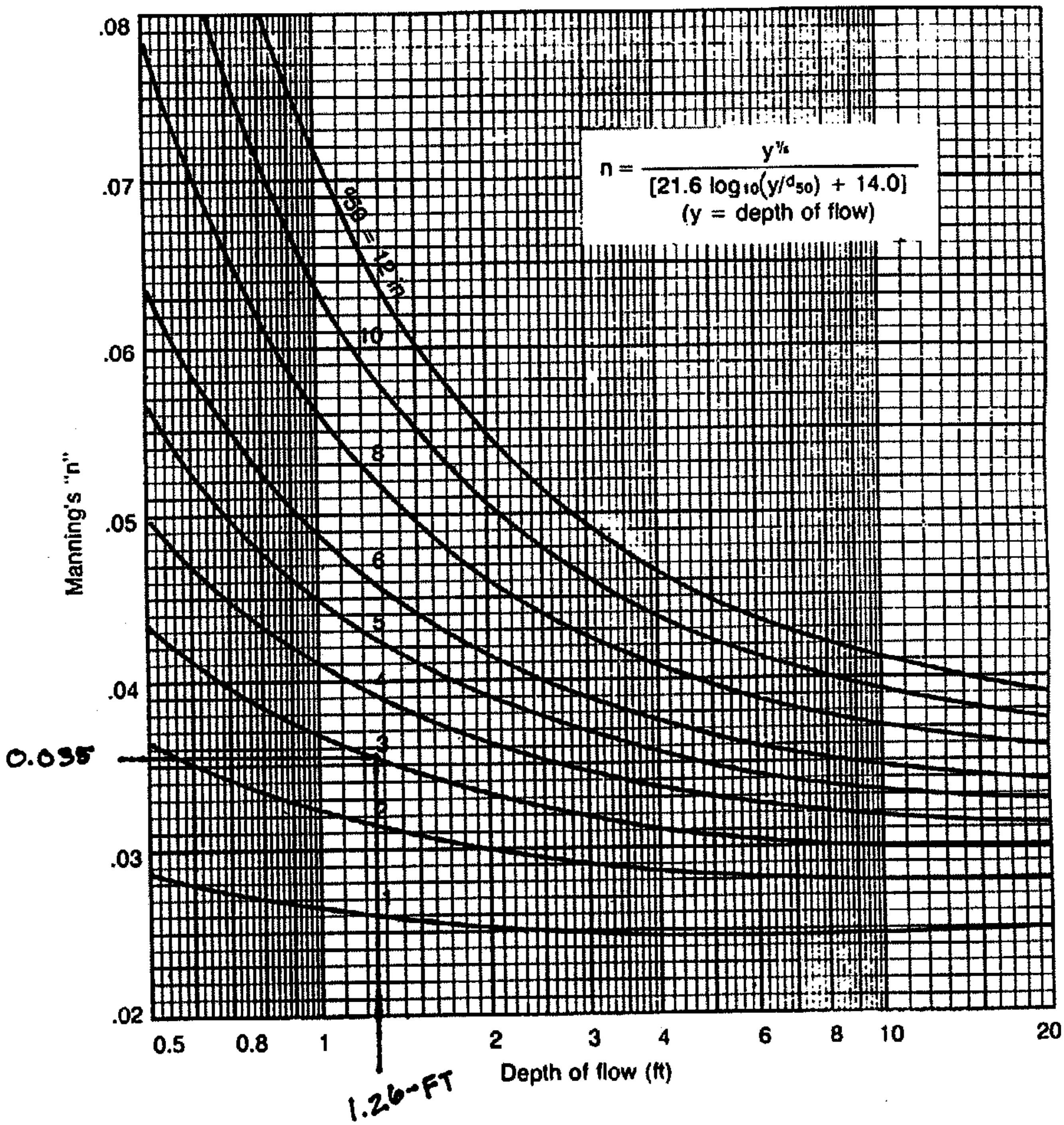
Attachment 4

References

Figure 5B.11
Determining "n" for Riprap Lined Channel using Depth of Flow
(USDA - NRCS)



Appendix H-11

RMU-2 South Ditch Design

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Infrastructure, environment, buildings

Imagine the result

Calculation Sheet

Client: CWM Chemical Services, LLC

Project Location: Model City, New York

Project: RMU-2 Design Calculations **Project No.: B0023725.2009**

Subject: Appendix H-11: RMU-2 South Ditch

Prepared By: GNG **Date: August 2009**

Checked By: BMS **Date: August 2009**

Reviewed By: PHB **Date: August 2009**

OBJECTIVE:

Demonstrate that the proposed cross-sectional geometry of the RMU-2 south ditch provides adequate hydraulic capacity to convey the estimated peak discharge from the 25-year, 24-hour storm. Demonstrate that stable hydraulic conditions exist in the RMU-2 south ditch.

REFERENCES:

1. RMU-2 Permit Drawing No. 7 entitled "Top of Vegetative Cover Grades", ARCADIS, August 2009.
2. Technical Release 55 "Urban Hydrology for Small Watersheds," Soil Conservation Service, June 1986.
3. HydroCAD Software Solutions, LLC, HydroCAD. Version 8.5. Computer Software, 2006. (Output attached).
4. "New York Guidelines for Urban Erosion and Sediment Control," August 2005.
5. Hydraulic Engineering Circular No.15 (HEC-15) "Design of Roadside Channels with Flexible Linings", U.S. Department of Transportation Federal Highway Administration, April 1988.

ASSUMPTIONS:

1. The RMU-2 south ditch is located along the southern edge of RMU-2 adjacent to Cell 19 and conveys stormwater runoff from portions of SLF 10 and RMU-2 to the V05 retention basin (Reference 1). The RMU-2 south ditch has a trapezoidal geometry with a 4-foot base width and sideslopes of 2H:1V on the southern edge and 0.25H:1V (MSE wall slope) on the northern edge. The minimum channel depth is 2-feet.
2. The invert slope of the RMU-2 south ditch is 0.3%.
3. The design storm is the 25-year, 24-hour event, which produces 4.0 inches of rainfall.
4. The RMU-2 south ditch is evaluated for two tributary watershed conditions. A partially vegetated RMU-2 cap condition (5.01 acres vegetated and 5.01 acres unvegetated) is intended to represent the increased flowrates from newly capped areas of RMU-2. For completeness, a fully vegetated (10.02 acres vegetated) final cover condition, which yields a lower flowrate, is also included in the evaluation.

Calculation Sheet

5. Runoff curve numbers for the tributary watershed to the RMU-2 south ditch includes 79 for capped areas with fully established vegetation, 91 for newly capped areas, and 89 for gravel roads and riprap-lined ditches (based on Reference 2 for Hydrologic Soil Group “C”).

6. The Manning “n” value for the RMU-2 south ditch is based on Reference 4 and assumes riprap with a D_{50} of 3 inches.

7. No minimum freeboard is required for the ditch due to the presence of grass (or otherwise stabilized) conditions adjacent to the edges of the RMU-2 south ditch (Reference 4).

8. To evaluate the stability of the riprap lining in the ditch, a shear stress approach is used assuming a worst-case scenario. The resultant shear stress is based on the peak flowrate for the 25-year, 24-hour design storm.

CALCULATIONS:

1. Estimated Peak Discharges

The tributary watershed that is used to design and evaluate the RMU-2 south ditch is depicted on the watershed map included in Attachment 1. Table 1 summarizes the runoff characteristics for the tributary watershed draining to the ditch and the 25-year, 24-hour estimated peak discharges under both partially vegetated and fully vegetated conditions.

Table 1 – Watershed Characteristics

Watershed Description	Watershed Area [acres]	Runoff Curve Number	Time of Concentration [min]	25-yr, 24-hr Estimated Peak Discharge [cfs]
Partially Vegetated Cap	16.74	84	40.5	27.9
Fully Vegetated Cap	16.74	80	40.7	23.7

Supporting output from HydroCAD is included in Attachment 2.

2. Resulting Hydraulic Conditions

Table 2 summarizes the resulting hydraulic conditions within the RMU-2 south ditch under both partially vegetated and fully vegetated conditions.

Table 2 – Flow Characteristics

Watershed Description	25-yr, 24-hr Estimated Peak Discharge [cfs]	Manning “n”	Minimum Channel Depth [in]	Flow Depth [in]	Flow Velocity [ft/s]
Partially Vegetated Cap	27.9	0.034	24	22	2.6
Fully Vegetated Cap	23.7	0.034	24	20	2.5

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Infrastructure, environment, buildings

Imagine the result

Calculation Sheet

A minimum channel depth of 24 inches for the RMU-2 south ditch provides adequate hydraulic capacity to convey the 25-year, 24-hour estimated peak discharge. Supporting output is included in Attachment 3.

3. Shear Stress Analysis for Riprap Ditch Lining

To calculate the maximum shear stress, τ_b, on the bed of a channel using Reference 5:

$$\tau_b(\frac{lb}{ft^2}) = (\gamma_w)(y)(i)$$

Where,
γ_w = Unit weight of water (62.4 lb/ft^3)
y = Depth of flow (ft) = 1.81 ft (partially vegetated condition)
y = Depth of flow (ft) = 1.65 ft (fully vegetated condition)
i = Bed slope (ft/ft)= 0.003 ft/ft

To calculate the allowable shear stress, τ_c, on the bed of the channel using Reference 5:

$\tau_c(\frac{lb}{ft^2}) = 4D_{50}$, where
D50 = 0.25 ft (Assumption 6)

The factor of safety is then determined by

Factor of Safety = $\frac{Allowable}{Maximum} = \frac{\tau_c}{\tau_b}$

To calculate the maximum shear stress, τ_m, on the sideslope of a channel using Reference 5:

$$\tau_m(\frac{lb}{ft^2}) = .75(\gamma_w)(y)(i)$$

Where,
γ_w = Unit weight of water (62.4 lb/ft^3)
y = Depth of flow (ft) = 1.81 ft (partially vegetated condition)
y = Depth of flow (ft) = 1.65 ft (fully vegetated condition)
i = Bed slope (ft/ft)= .003 ft/ft

To calculate the allowable shear stress, τ_s, on the sideslope of the channel using Reference 5:

$$\tau_s(\frac{lb}{ft^2}) = (\tau_c)\left(\sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}}\right)$$

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Infrastructure, environment, buildings

Imagine the result

Calculation Sheet

θ = side angle of the channel = $\tan^{-1}$ (1/2) = 26.6 degrees
ϕ = angle of repose of rip rap = 40 degrees

The shear stress analysis for the bed and sideslopes of the channel is summarized in Table 3 below.

Table 3 - Shear Stresses on Bed and Sideslopes

Watershed Description	Channel Surface	Flowrate (cfs)	Flow Depth (ft)	Resulting Shear Stress (lb/ft^2)	Allowable Shear Stress (lb/ft^2)	Factor of Safety
Partially Vegetated	Bed	27.9	1.81	0.34	1.00	2.95
	Sideslopes	27.9	1.81	0.25	0.38	1.49
Fully Vegetated	Bed	23.7	1.65	0.31	1.00	3.23
	Sideslopes	23.7	1.65	0.23	0.38	1.63

As indicated in Table 3, the RMU-2 south ditch is hydraulically stable. Supporting output is included in Attachment 3.

SUMMARY:

Based on the above calculations, the RMU-2 south ditch provides sufficient capacity to convey the estimated peak flow from the 25-year, 24-hour design storm. The riprap lining in the RMU-2 south ditch is hydraulically stable while conveying the peak flowrate from the 25-year, 24-hour storm.

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Attachment 1

Watershed Map

ARCADIS	
SHEET TITLE: RMU-2 SOUTH DITCH WATERSHED MAP PROJECT TITLE: RMU-2 PERMIT CALCULATION CWM CHEMICAL SERVICES PROJECT LOCATION: MODEL CITY, NEW YORK PROJECT NO: B0023725.2009 DATE: AUGUST 2009 BY: GNG FILE NO.: 23725.PRO	Figure 1

ARCADIS

Attachment 2

HydroCAD Output

Summary for Subcatchment 1S: RMU-2 South Ditch: Partially Vegetated Cap

Runoff = 27.94 cfs @ 12.37 hrs, Volume= 3.287 af, Depth= 2.37"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type II 24-hr 25-year Rainfall=4.00"

	Area (ac)	CN	Description
*	4.520	79	SLF 10
	0.340	89	Gravel roads, HSG C
*	0.260	89	SLF 10 Ditch
*	5.010	91	Unvegetated RMU-2 Cap
*	5.010	79	Vegetated RMU-2 Cap
*	0.573	89	RMU-2 Perimeter Ditch
*	0.316	89	RMU-2 Ditch
*	0.605	89	Perimeter Road
	16.634	84	Weighted Average
	16.634		Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
36.3					**Direct Entry, SLF 10 Ditch Tc**
4.2	633		2.54		**Direct Entry, RMU-2 South Ditch**
40.5	633	Total			

Summary for Subcatchment 2S: RMU-2 South Ditch: Fully Vegetated Cap

Runoff = 23.73 cfs @ 12.38 hrs, Volume= 2.830 af, Depth= 2.04"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type II 24-hr 25-year Rainfall=4.00"

	Area (ac)	CN	Description
*	4.520	79	SLF 10
	0.340	89	Gravel roads, HSG C
*	0.260	89	SLF 10 Ditch
*	10.020	79	Vegetated RMU-2 Cap
*	0.573	89	RMU-2 Perimeter Ditch
*	0.316	89	RMU-2 Ditch
*	0.605	89	Perimeter Road
	16.634	80	Weighted Average
	16.634		Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
36.3					**Direct Entry, SLF 10 Ditch Tc**
4.4	633		2.40		**Direct Entry, RMU-2 South Ditch**
40.7	633	Total			

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Attachment 3

Hydraulic Analysis

Project: RMU-2 Design
Project No.: B0023725.2009
Subject: RMU-2 South Ditch

Prepared By: GNG
Date: August 2009
Checked By:____
Date:____

Riprap Lining Partially Vegetated Condition

Channel Design (Input)	
Flow Capacity (cfs)	27.94
Base Width (ft)	4.00
Left Side Slope (x:1)	2.00
Right Side Slope (x:1)	0.25
Bed Slope	0.003
Minimum Riprap D50, (in.)	3.0
Maximum Allowable Shear Stress on Bed (psf)	1.00
Maximum Allowable Shear Stress on Sideslopes (psf)	0.38
Manning "n"	0.034

Flow Conditions (Output)	
Flowrate from Manning Equation (cfs)	27.94
Required Flow Depth (ft)	1.81
Resulting Flow Velocity (ft/s)	2.56
Resulting Flow Width at Top (ft)	8.07
Resulting Flow Area (ft^2)	10.91
Resulting Wetted Perimeter (ft)	9.91
Resulting Hydraulic Radius (ft)	1.10
Resulting Shear Stress on Bed (psf)	0.34
Resulting Shear Stress on Sideslopes (psf)	0.25
Channel Dimensions	
Channel Depth (ft)	2.00
Resulting Freeboard (ft)	0.19
Shear Stress Factor of Safety (Bed)	2.95
Shear Stress Factor of Safety (Sideslope)	1.49

Project: RMU-2 Design
Project No.: B0023725.2009
Subject: RMU-2 South Ditch

Prepared By: GNG
Date: August 2009
Checked By:____
Date:____

Riprap Lining Fully Vegetated Condition

Channel Design (Input)	
Flow Capacity (cfs)	23.73
Base Width (ft)	4.00
Left Side Slope (x:1)	2.00
Right Side Slope (x:1)	0.25
Bed Slope	0.003
Minimum Riprap D50, (in.)	3.0
Maximum Allowable Shear Stress on Bed (psf)	1.00
Maximum Allowable Shear Stress on Sideslopes (psf)	0.38
Manning "n"	0.034

Flow Conditions (Output)	
Flowrate from Manning Equation (cfs)	23.73
Required Flow Depth (ft)	1.65
Resulting Flow Velocity (ft/s)	2.45
Resulting Flow Width at Top (ft)	7.72
Resulting Flow Area (ft^2)	9.69
Resulting Wetted Perimeter (ft)	9.40
Resulting Hydraulic Radius (ft)	1.03
Resulting Shear Stress on Bed (psf)	0.31
Resulting Shear Stress on Sideslopes (psf)	0.23
Channel Dimensions	
Channel Depth (ft)	2.00
Resulting Freeboard (ft)	0.35
Shear Stress Factor of Safety (Bed)	3.23
Shear Stress Factor of Safety (Sideslope)	1.63

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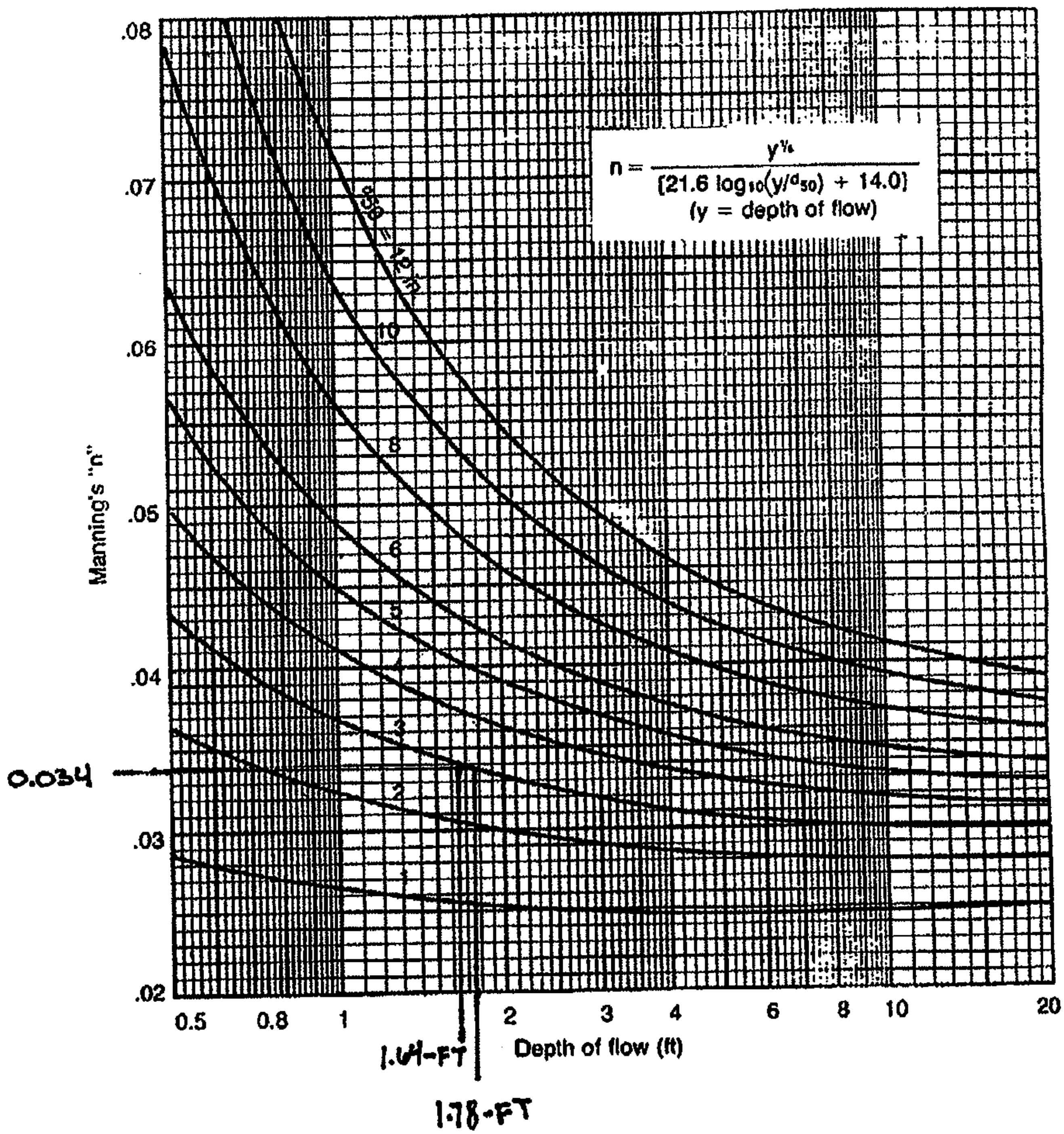
Attachment 4

References

Figure 5B.11
Determining "n" for Riprap Lined Channel using Depth of Flow
(USDA - NRCS)



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Appendix H-12

Downchute Pipe Thrust Block Design

Imagine the result

Calculation Sheet

Client: CWM Chemical Services, LLC

Project Location: Model City, New York

Project: RMU-2 Design Calculations **Project No.: B0023725.2011**

Subject: Appendix H-12: Downchute Pipe Thrust Block Design

Prepared By: NWF **Date: February 2013**

Reviewed By: PTO **Date: February 2013**

Checked By: BMS **Date: February 2013**

OBJECTIVE:

Demonstrate the proposed thrust blocks are adequate in restraining the 34-inch-diameter HDPE downchute header pipes at the western edge and northeast corner of RMU-2.

REFERENCES:

1. RMU-2 Permit Drawing No. 24 entitled “Surface Water Management Details”, ARCADIS, February 2013.

2. Thrust Restraint Design for Ductile Iron Pipe, DIPRA, Sixth Edition, 2006.

3. U.S. Army Corps of Engineers, Revision of Thrust Block Criteria in TM 5-818-5/AFM 88-10, Vol. 5, Appendix C.

ASSUMPTIONS:

1. As depicted in Reference 1, 34-inch-diameter header pipes, installed at the base of the RMU-2 Mechanically Stabilized Earth (MSE) wall, convey stormwater flow from a series of downchute pipes that provide stormwater drainage for the stormwater diversion berms and perimeter ditches atop the MSE wall of RMU-2. As such the flows from the downchute pipes apply hydraulic pressure on the header pipe and therefore a concrete thrust block will be used to anchor the header pipe.

2. The thrust block is considered acceptable if the bearing capacity of the soil below the thrust block is not exceeded and the frictional forces between the thrust block and the underlying soils can resist the hydrostatic forces applied to the header pipe by the downchute pipes while maintaining a factor of safety greater than or equal to 1.5. The ability of the thrust block to resist sliding is based on the soil coefficient of friction and the thrust block weight.

3. The assumed unit weight of concrete for the thrust block is 150 lb/ft^3 based on Reference 3. Based on Reference 3, the coefficient of friction for concrete cast on soil (sandy/silty) is 0.45.

4. For calculation purposes, the weight of the cover soils over the proposed thrust block is not considered. Further, the stabilizing effect of soil to the outside of the thrust block is not considered. Collectively, these assumptions yield a conservative thrust block design.

Calculation Sheet

5. The soil bearing capacity of the native soil below the proposed thrust block is assumed to be 3,000 lb/ft^2 and assumes sandy/silty soil (Reference 2).

6. For calculation purposes, the downchute pipes are conservatively assumed to be completely filled with water up to the pipe inlets. Further, the discharge end of the downchute header pipe is assumed to be plugged. This yields the maximum possible head (and therefore thrust) on the thrust block. The maximum water surface elevations for the perimeter ditch outlet pipes and surface water diversion berm drainage pipes are 348.07 ft and 371.08 ft, respectively. The header pipe sits at an elevation of 316.48 ft. Therefore the maximum hydrostatic head acting on the header pipe for each perimeter ditch outlet pipe and surface water diversion berm drainage pipe is 31.59 ft and 56.60 ft, respectively. Because of the large diameter of the header pipe, a complete blockage of the pipe that would then allow such high heads to build in the downchute pipes is not considered likely, leading to a conservative thrust block design.

7. For calculation purposes, the thrust block is assumed to have a length of 15 ft (Reference 1). The thrust block is assumed to be cast around the header pipe such that a minimum of 12 inches of concrete will be below, above, and to the outside of the header pipe. A diagram of the assumed header pipe and thrust block dimensions is included as an attachment to this calculation.

8. The parameters used in the analysis are summarized in Table 1 below:

Table 1-Calculation Variables

Variable	Value
Cross Sectional Area of Downchute Pipe Interior, A	3.14 ft^2
Cross Sectional Area of Header Pipe Interior, A'	6.29 ft^2
Head, h	31.59 ft, 56.60 ft
Angle of Resultant Force Acting on Thrust Block, θ	14°
Minimum Acceptable Factor of Safety	1.50
Unit Weight of Water, γ_w	62.4 pcf
Unit Weight of Concrete, γ_c	150 pcf
Coefficient of Friction for Concrete Cast on Soil, C_f	0.45

CALCULATIONS:

1. Maximum Resultant Force Acting on Thrust Block

This analysis is performed by evaluating the hydrostatic forces acting on the downchute header pipe from three downchute pipes at each location (1 surface water diversion berm drainage pipe and 2 perimeter ditch outlet pipes). Runoff is conveyed through the downchute pipes into the HDPE header pipe which is capped at the upstream end (Reference 1). Because the intersection of the downchute pipes and the header pipe is essentially a 90 degree tee, the thrust imparted to the header pipe is equal to the static pressure in each pipe multiplied by the cross sectional area of the respective downchute pipe. The resultant thrust force calculation is as follows:

Imagine the result

Calculation Sheet

$F_R = P * A = \gamma_w * h * A$

Where,

F_R = Resultant Force (lbs)
P = Pressure (lb/ft^2)
γ_w = Unit Weight of Water (lb/ft^3)
h = Head (ft)
A = Cross Sectional Area (ft^2)

Because the header pipe contains flow from three separate downchute pipes, the total force from all three pipes must be combined to calculate the resultant force on the header pipe. As indicated in Attachment 1, the resultant force acting on the header pipe from the combination of pipes is approximately 23,500 lbs.

The resultant force is broken into F_x and F_y components to determine the horizontal and vertical forces acting on the thrust block from the downchute pipes. The F_x and F_y components of the thrust force are calculated as follows:

$F_x = F_R * Sin\theta$
$F_y = F_R * Cos\theta$

Where,

F_R = Resultant Thrust Force (lbs)
θ = Angle of the Resultant Force with Respect to Vertical (degrees)

Based on the equations above, the thrust forces acting in the horizontal (F_x) and vertical (F_y) plane are 5,690 lbs and 22,800 lbs, respectively. A diagram of the pipe and thrust block is included in the attachments to supplement the equations provided above.

2. Frictional Force Calculations

As indicated below, the frictional force or sliding force is calculated by determining the friction force at the bottom of the concrete block, where C_f is the coefficient of friction for concrete directly on soil and W_{TB} is the weight of the concrete thrust block. F_f is the frictional force acting on the thrust block preventing it from sliding. The thrust block is considered acceptable if the frictional force between the block and underlying soils is greater than or equal to 1.5 times the lateral force that is exerted by the downchute pipes. The calculation below is used to determine the frictional force:

$F_f = C_f * W_{TB}$

Calculation Sheet

Where,

F_f = Frictional Force (lbs)

W_{TB} = (150 lb/ft^3) $\left(\left[\left(\frac{1}{4}\pi * W^2\right) + (W * H')\right] * L\right) - \left[\frac{\pi * d^2}{4} * L\right]$ = Weight of Concrete Thrust Block (lbs)

C_f = Coefficient of Friction (Assumption 3)

A summary of the results of the frictional force calculation is shown in Table 2 below. A free body diagram showing forces acting on the thrust block is included in the attachments to supplement the equations provided above.

Table 2- Frictional Force Calculations

Calculated Frictional Force of Thrust Block (lbs)	Lateral Force Exerted on Thrust Block (ft^2)	Factor of Safety
9,180	5,690	1.6

As stated in Assumption 4, the additional stabilizing effect of soil to the outside of the thrust block is not considered. This increased resistance to sliding would further increase the factor of safety.

3. Bearing Capacity Assessment

The equation below is used as a check to determine if the bottom (bearing) area of the proposed thrust block is adequately sized to withstand the normal force of the thrust block without exceeding the bearing capacity of the underlying soils. If the bearing area of the proposed thrust block is greater than the minimum acceptable area (A_{TB}) then the thrust block is considered acceptable. As indicated below, the minimum acceptable area of the thrust block is calculated using the vertical component of the thrust imparted on the block, the weight of the block itself, the allowable bearing capacity of the soil, and a factor of safety of 1.5. The calculation to determine the minimum acceptable area of the thrust block is as follows:

$$A_{TB} = \frac{F_N * FS}{q_s}$$

Where,

A_{TB} = Minimum Acceptable Bottom Area of Thrust Block
q_s = Allowable bearing capacity of the soil
F_N = Normal Force
FS = Factor of Safety

Based on the calculation above, the minimum acceptable area of the thrust block is equal to 21.6 ft^2. A

Calculation Sheet

summary of the proposed thrust block dimensions, and minimum acceptable area is presented in Table 3 below.

Table 3- Bearing Capacity Calculations

Proposed Bearing Area of Thrust Block (ft^2)	Minimum Acceptable Bearing Area of Thrust Block (ft^2)	Factor of Safety
57.5	21.6	2.7

SUMMARY:

The proposed thrust block is sufficient to anchor the downchute header pipe both horizontally and vertically while maintaining a factor of safety of 1.5 or greater for the worst-case scenario in which the downchute pipes are completely filled with water to the inlets and the discharge end of the header pipe is assumed to be plugged.

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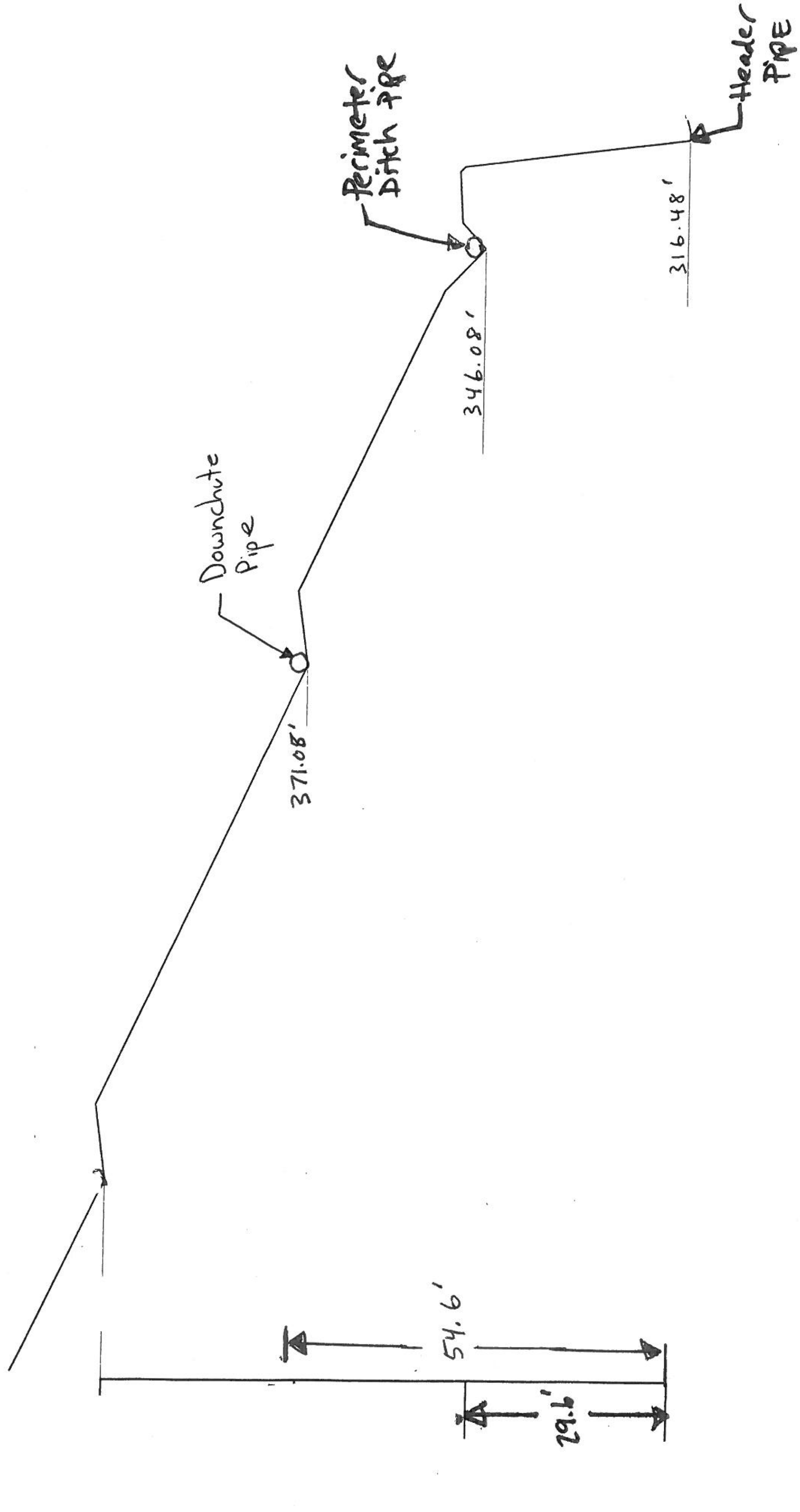
Attachment 1

Free Body Diagram and Calculations


Header Pipe
Perimeter Ditch Pipe
316.48'
346.08'
Downchute Pipe
371.08'
54.6'
29.6'

RESULTANT FORCE ON THRUST BLOCK :

$F = \gamma h A$

$\gamma = 62.4 \ lb/ft^3$

$h =$ head

$A =$ cross sectional Area (ft^2)

$F_{\text{perim Ditch outlet pipe}} = \gamma h A = (62.4 \ lb/ft^3)(31.59 \ ft)(\pi (1')^2)$

$= 6,200 \ lbs \ (2)$ (Two pipes)

$= 12,400 \ lbs$

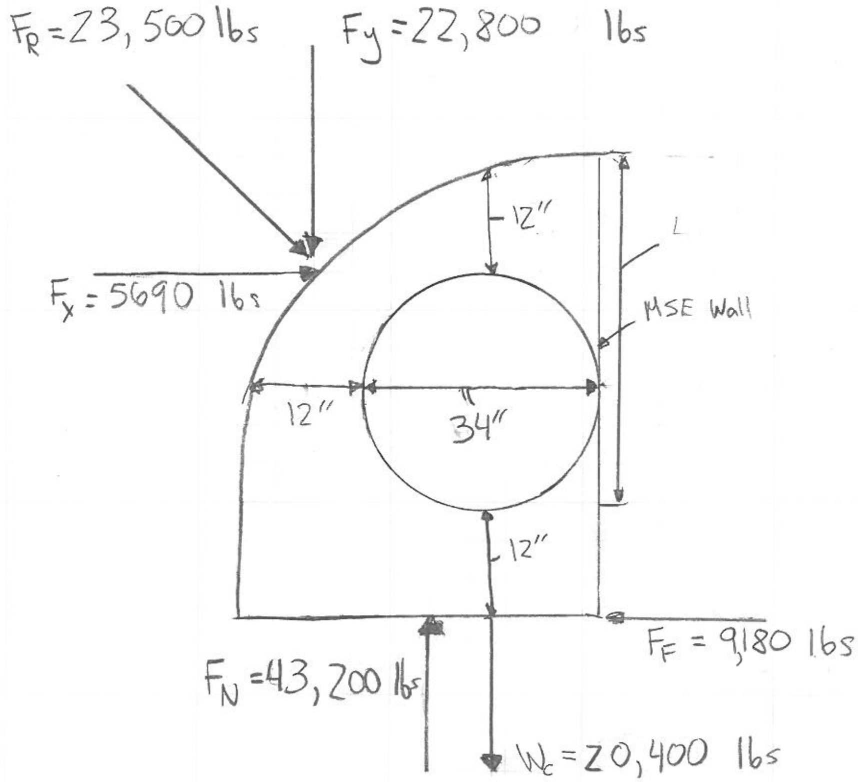
$F_{\text{Downchute}} = \gamma h A = (62.4 \ lb/ft^3)(56.6 \ ft)(\pi (1')^2)$

$= 11,100 \ lbs$

$F_R = 23,500 \ lbs$

$F_x = F_R \sin \theta = 23,500 \ \sin(14°) = 5,690 \ lbs$

$F_y = F_R \cos \theta = 23,500 \ lbs \ \cos(14°) = 22,800 \ lbs$



$F_F = W_{TB} \cdot C_F$

$C_F = 0.45$

$W_{TB} = \gamma \, vol = (150 \text{ lbs/ft}^3)\left\{\left[\frac{1}{4}\pi\left(\frac{12''+34''}{12}\right)^2\right] + \left[\frac{(46'')(12'')}{144}\right](15\text{ft}) - \left[\frac{\pi(2.83^2)}{4}(15\text{ FT})\right]\right\}$

$W_{TB} = (150 \text{ lbs/ft}^3)(230 \text{ ft}^3 - 94 \text{ ft}^3) = 20,400 \text{ lbs}$

$F_F = (0.45)(20,400 \text{ lbs}) = 9,180 \text{ lbs}$

Bearing Capacity Calcs

Minimum Bottom Area of Thrust Block

$$\text{Min } A_{TB} = F_S(F_n) / q_s = \frac{(1.5)(20,400 + 22,800)}{3,000 \text{ lbs/ft}^2}$$

$$\text{Min } A_{TB} = 21.6 \text{ Ft}^2$$

$$\text{Actual Area} = (15 \text{ Ft})\left(\frac{12'' + 34''}{12}\right) = 57.5 \text{ Ft}^2$$

$$57.5 > 21.6$$

$\therefore$ Area of thrust block is adequate

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Attachment 2

References

C-1. General. Thrust forces occur in waterlines when the pipeline changes directions, changes sizes, or stops. Thrust blocks or restrained joints are used to resist thrust forces. The geotechnical parameters used herein should be developed by a geotechnical engineer.

C-2. Thrust Forces. The magnitude of the thrust force may be calculated by:

$T = 2PA \sin \varnothing/2$ at bends;

or by:

$T = PA$ at deadends, branches, or tees.

Where,

T = thrust force in pipe,
P = internal pressure of pipe,
A = internal cross-sectional area of pipe, and
$\varnothing$ = angle of deflection of bend.

These are shown in Figures C-1, C-2 and C-3.

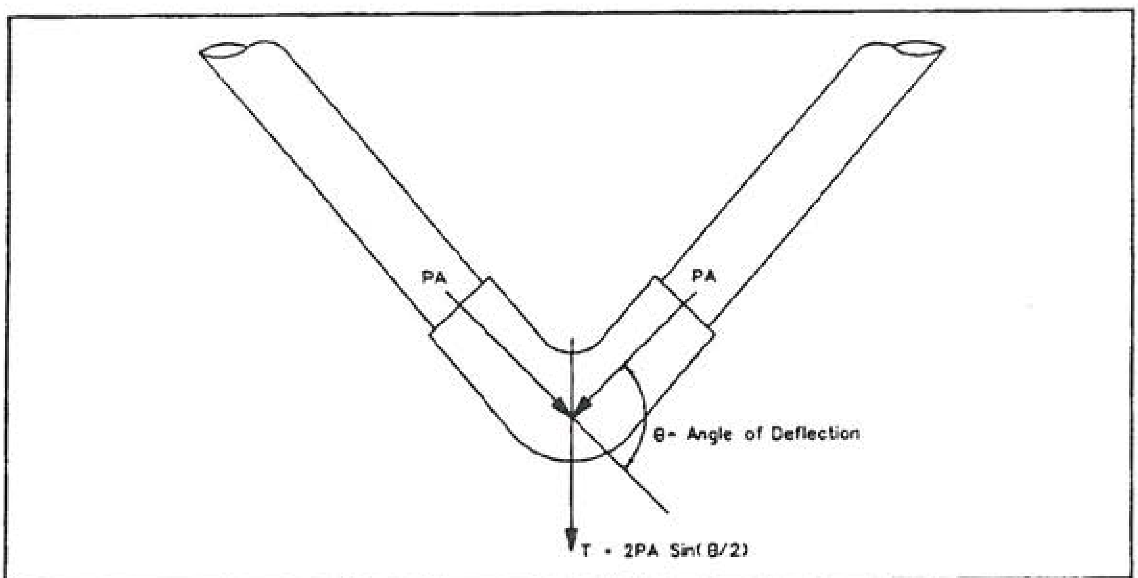

Figure C-1

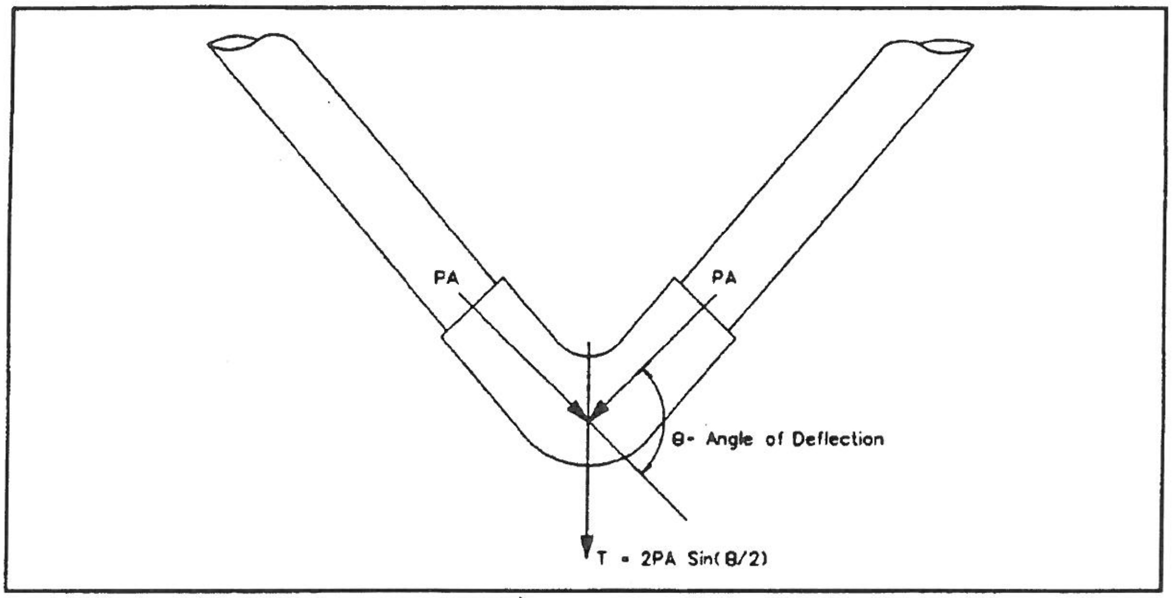

Figure C-4

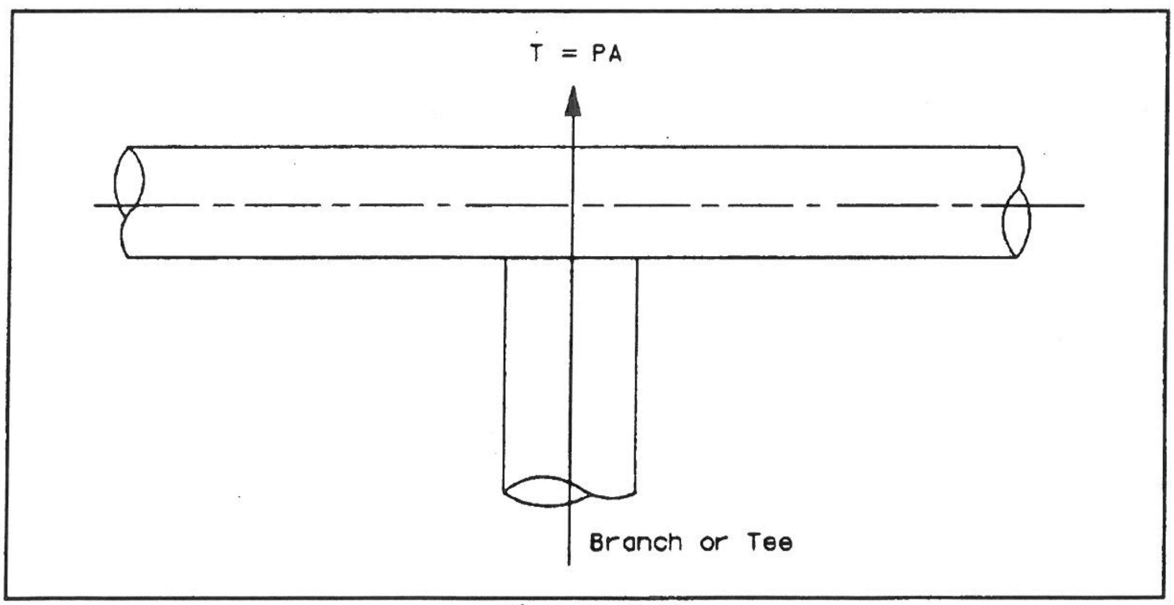

Figure C-5

The following are general criteria for bearing block design.

- Bearing surface should, where possible, be placed against undisturbed soil. Where it is not possible, the fill between the bearing surface and undisturbed soil must be compacted to at least 90% Standard Proctor density.
- Block height (h) should be equal to or less than one-half the total depth to the bottom of the block, (H_t), but not less than the pipe diameter (D′).
- Block height (h) should be chosen such that the calculated block width (b) varies between one and two times the height.

The required bearing block area is

$$A_b = hb = \frac{S_f T}{S_b}$$

Then, for a horizontal bend,

$$b = \frac{S_f\ 2\ PA \sin(\Theta/2)}{h\ S_b} \qquad \textbf{(1)}$$

where S_f is a safety factor (usually 1.5 for thrust block design). A similar approach may be used to design bearing blocks to resist the thrust forces at tees, dead ends, etc. Typical values for conservative horizontal bearing strengths of various soil types are listed in Table 1.

In lieu of the values for soil bearing strength shown in Table 1, a designer might choose to use calculated Rankine passive pressure (P_p) or other determination of soil bearing strength based on actual soil properties.

Gravity thrust blocks may be used to resist thrust at vertical down bends. In a gravity block, the weight of the block is the force providing equilibrium with the thrust force. The design problem is then to calculate the required volume of the thrust block of a known density. The vertical component of the thrust force in Figure 6 on page 7 is balanced by the weight of the block.

It can easily be shown that $T_y = PA \sin \Theta$. Then the required volume of the block is

$$V_g = \frac{S_f\ PA \sin \Theta}{W_m} \qquad \textbf{(2)}$$

where W_m = density of the block material. Here, the horizontal component of the thrust force

$$T_x = PA\ (1 - \cos \Theta)$$

must be resisted by the bearing of the right side of the block against the soil. Analysis of this aspect will follow like the above section on bearing blocks.

Calculations of V_g and T_x for orientations other than when one leg is horizontal should reflect that specific geometry.

Table 1

Horizontal Bearing Strengths

Soil	*Bearing Strength S_b (lb/ft²)
Muck	0
Soft Clay	1,000
Silt	1,500
Sandy Silt	3,000
Sand	4,000
Sandy Clay	6,000
Hard Clay	9,000

*Although the above bearing strength values have been used successfully in the design of thrust blocks and are considered to be conservative, their accuracy is totally dependent on accurate soil identification and evaluation. The ultimate responsibility for selecting the proper bearing strength of a particular soil type must rest with the design engineer.

TABLE C-1

Friction Coefficient for Concrete Cast on Soil
(reference 4)

Interface Materials	Friction Coefficient, f
Mass concrete on the following foundation materials:	
Clean sound rock	0.70
Clean gravel, gravel—sand mixtures, coarse sand	0.55 to 0.60
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55
Clean fine sand, silty or clayey fine to medium sand	0.35 to 0.45
Fine sandy silt, nonplastic silt	0.30 to 0.35
Very stiff and hard residual or preconsolidated clay	0.40 to 0.50
Medium stiff and stiff clay and silty clay	0.30 to 0.35

C-4.2. The size of thrust block for downward directed thrust is calculated by;

$$A_{TB} \geq F_S T_Y / q_S$$

where;

A_{TB} = bottom area of thrust block,
T_y = vertical component of thrust force,
q_S = allowable bearing capacity of soil, and
F_S = Factor of Safety.

C-4.3. There is also a horizontal component of thrust (T_x) in vertical bends. The sizing of thrust block for the horizontal component is calculated by the same formula used for horizontal bends, except the term T is replaced by T_x = 2PA Sin 0/2 Cos 0.

C-4.4. These are shown in Figures C-4, C-5, C-6 and C-7.

C—5. Restrained Joints. There are several approaches to this. They all calculate the length of pipe to be restrained on both sides of the joint. The length to be restrained may be determined by;

$$L \geq F_S(PA \tan \bullet/2)/(F_f + 0.5\ R \bullet_s\ Z\ K_p\ D_o)$$

where;