Attachment 1
Black and Veatch Calculation Package

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CALCULATION COVER SHEET

Client Name:	Unistar	Project No.:	161642
Project Name:	Bell Bend Nuclear Power Plant	Calculation No.:	161642.51.1512
Calculation Title:	ESWEMS Pond Spillway Design	File No.:	18.9220
		Page	1 of 44

Calculation Type	☐ Preliminary	☒ Final	
Seismic Classification	☒ I	☐ II	
Quality Classification	☒ S	☐ R	☐ G

Objective 1. To determine if the spillway which was preliminarily designed to pass 30 cfs with approximately 1 foot of head, was adequate to pass the PMP, which was determined by others. 2. To determine if a 6-inch diffuser block is necessary for the spillway. 3. To check for the potential for erosion of the rip-rap at the end of the apron while passing the PMP.

Unverified Assumptions Requiring Subsequent Verification

No.	Assumption	Verified By	Date
	N/A		

Refer to page _____ of this calculation for additional assumptions.

This Section Used for Computer Calculations

Software Program Name/Number: **Bentley FlowMaster** Version: Service Pack 3

Number of Pages: N/A

Evidence of or reference to computer program verification, if applicable:
See Appendix 2 for hand calculation of verification of Bentley FlowMaster.

Bases or reference thereto supporting application of the computer program to the physical problem:
N/A

Review and Approval

Rev.	Prepared By: Print and Sign	Prepared By: Date	DVR No.*	Approved By: Print and Sign	Approved By: Date
0	Steven W. Williams	July 1st, 2009	DVR-0039	Richard H. Gomy	July 1, '09

Revision Description

*Indicate Design Verification Review (DVR) Checklist number.
This calculation supersedes Calculation Number: N/A
This calculation is superseded by Calculation Number: N/A

1.0 Purpose and Scope

1.1 Determine if the spillway, which was preliminarily designed to pass 30 cfs with approximately 1 foot of head, was adequate to pass the PMP, which was determined by others. The spillway has a 6-foot bottom width and 3H:1V side slopes.

1.2 The initial design has a 6-inch high diffuser block at the end of the apron. The second purpose of the calculation is to determine if the diffuser block is necessary, and if it was adequately sized.

1.3 The third purpose is to check for the potential for erosion of the rip-rap at the end of the apron while passing the PMP. The rip rap has approximately a 100 pound 50 percent size.

2.0 Reference Materialss

2.1 References

2.1 Finnemore, E. John., Franzini, Joseph B. Fluid Mechanics with Engineering Applications Tenth Edition. McGraw-Hill, 2002.

2.2 Michael R. Civil Engineering Reference Manual. Eleventh Edition. Professional Publications. 2008.

2.2 Appendices

1. Design Input References for Hydraulic Calculations - 8 pages
2. Bentley FlowMaster Calculations and hand calculation check - 4 pages
3. USACE Hydraulic Design Criteria, Sheet 712-1, Stone Stability - 7 pages

2.3 Attachments

A. PCR Calculation 07-3891. PMF for Bell Bend NPP ESWEMS Pond. 10/01/08 - 9 pages
B. Lag Time Calculation - 3 pages
C. Elevation - Storage Curve - 2 pages
D. HEC-HMS Output - 3 pages

3.0 Conclusions

3.1 The 72 hour PMP determined by PCR is 37.77 inches (3.15 feet). The normal pond level is at elevation 669.0, and the spillway crest is at elevation 672, therefore, the PMP barely overtops the spillway crest. PCR's calculations indicate that the maximum outflow is 0.71 cfs (Attachment 1); therefore the preliminary design of 30 cfs is adequate.

3.2 Calculations indicate that the hydraulic jump occurs on the 3H:1V slope, before it reaches the apron, therefore, the diffuser block is not required. When passing the PMP, the maximum water depth on the apron is 0.25 feet, and the velocity is 0.42 ft/s (Appendix 2). Although a diffuser block is not required, it is considered good practice, and the design is acceptable "as-is".

3.3 Rip-rap with a 50 percent size of 100 pounds, equivalent spherical size of 1.05 feet, is adequate for turbulent flow of approximately 9 ft/s (Page 8). The velocity on the apron is 0.42 ft/s, therefore, the rip-rap is adequate and will not experience erosion while passing the PMP.

4.0 Procedure/Methodology of Design

4.1 Design of Spillway

The spillway used was originally designed for a flow of 30 cfs. Using the PCR calculation, PMF for Bell Bend NPP ESWEMS Pond, the actual flow rate coming over the spillway was compared to the design flow rate to determine if the spillway has adequate capacity.

4.2 Hydraulic Jump

To determine if a diffuser block is required it must be determined if the energy is dissipated on the apron before reaching the rip-rap lined channel. The dissipation of energy occurs during a hydraulic jump. The principle of momentum was used to determine the elevation to which a hydraulic jump would occur. The principle of momentum is as follows:

Step 1
Determine the normal depth on the spillway and the apron using Manning's Equation, found below.

$Q = 1.49 / n * A * R^{2/3} * S^{1/2}$ (Reference 2.1, page 412)

Where:

Q = flow rate, cfs
n = manning's roughness coefficient
A = flow area, ft^2
A = y * (b+Z*y) (See Appendix 1)
y = normal depth
B = base of channel
Z = side slopes of channel = 3 ft/ft
R = hydraulic radius, A/P
P = wetted perimeter, ft
$P = B + 2 * y * (1 + Z^2)^{1/2}$ (See Appendix 1)
S = slope, ft/ft

Step 2
A hydraulic jump will occur only when flow goes from supercritical to subcritical flow. To determine this the critical depth for the channel was calculated in FlowMaster and compared with the spillway and apron normal depths.

Step 3
Once it was determined that a hydraulic jump occurs it is now necessary to find the conjugate depth for the normal depth on the spillway using the momentum principle.

$F_1 - F_2 = \gamma * (Q^2/g/A_2 - Q^2/g/A_1)$ (Reference 2.1)

Where:

F = the force of the structure on the water in the horizontal direction, $\gamma * z * A$
z = depth to centroid, ft
γ = specific weight of water, lb/ft^3
A = flow area, ft^2
Q = flow rate, ft^3/sec

z, depth to centroid, of a trapezoidal channel is determined as follows:

The centroid of each section of the trapezoidal channel must be determined and then an area weighted average calculated.

Rectangular Section:

$z_r = 1/2 * y$ (Reference 2.1, page 738)
$A_r = B * y$

Triangular Sections

$z_t = 1/3 * y$ (Reference 2.1, page 738)

$A_t = 3 * y^2$

Total Trapezoidal Channel

$z = (z_r * A_r + z_t * A_t) / (A_r + A_t)$ (Reference 2.2, page 42-1)

Step 4
A hydraulic jump will occur either upstream or downstream of the of the slope transition. If the normal depth on the apron is less than the conjugate depth of the spillway an M3 water surface profile will exist and the hydraulic jump will occur downstream of the transition. If this is the case than it is necessary to calculate the conjugate depth of the apron.

If the normal depth on the apron is greater than the conjugate depth of the spillway an S1 water surface profile will exist and the hydraulic jump will occur upstream of the transition. (Reference 2.1, page 466, Figure 10.26)

The normal depth of the apron was compared with the spillway conjugate depth to determine if it was necessary to calculate the total length of the jump (gradually varied flow section plus the jump).

Step 5
If a M3 water surface profile exists it is necessary to calculate the total length of the jump. Otherwise this step can be skipped. It is important that the hydraulic jump occur on the apron to avoid scouring in the rip-rap lined channel. To determine if the jump occurs on the apron, the length of the jump, including the gradually-varied flow section, was calculated with the following equation.

Gradually Varied Flow
$L = (E_1 - E_2) / (S_e - S_0)$ (Reference 2.1, page 466)

Where:

E1 = Energy on spillway, $y1 + V1^2/(2*g)$
- y1 = normal depth on spillway
- V1 = Velocity on spillway (determined from FlowMaster)
- g = gravity constant

E2 = Energy from conjugate depth on Apron, $y4 + V4^2/(2*g)$
- y4 = conjugate depth for apron normal depth
- V4 = Velocity at this depth (determined using continuity equation Q = V * A
- g = gravity constant

S_e = Slope of energy grade line, $S_e = ((n * \underline{V}) / (1.49 * \underline{R}_h^{2/3}))^2$
- n = manning's roughness coefficient
- $\underline{V}$ = average velocities of above
- $\underline{R}_h$ = average hydraulic radius with above depths

S_0 = Slope of Apron

Jump
The length of a jump is difficult to predict. A good approximation for jump length is about 5 * y4. (Reference 2.1, page 464) This approximation will be used.

4.3 Erosion in Rip-Rap Lined Channel

For erosion to occur in a rip-rap lined channel, the velocity must be greater than the velocity it takes to move the rock. To determine the velocity at which the rip-rap will move, the following method was used. (See Appendix 3)

$V = c * (g_s - g_s/g_w)^{0.5} * D_{50}^{0.5}$

Where:

c = turbulence coefficient, 0.86 high turbulence, 1.2 low turbulence
g_s = gamma stone, 165 pcf

g_w = gamma water, 62.4 pcf

D_{50} = Diameter of rock, ft

$D = (6 * W50/ (\pi * g_s))^{0.33}$

W_{50} = weight of rip-rap, lbs

$\pi = 3.141593$

Compare the velocity on the apron with the velocity calculated here. If the apron velocity is less than the calculated velocity, no erosion will occur.

5.0 Assumptions

The slope on the apron is assumed to be 0.0001 ft/ft to represent flat conditions.

6.0 Definitions of Units and Constants

See body of calculations.

7.0 Analysis/Solution

7.1 Design of Spillway

Since the probable maximum flow out of the pond is 0.71 cfs (Attachment 1, page 18), the spillway has sufficient capacity

7.2 Hydraulic Depth

Step 1

The normal depth of the spillway and apron were calculated using Bentley FlowMaster (See Appendix 2). A verification of the output of FlowMaster is also included in Appendix 2. The input is as follows:

$Q := 0.71 \frac{ft^3}{s}$ (Attachment 1, page 16)

$n := 0.013$ (Reference 2.1, page 412, concrete)

$B := 6ft$

$Z := 3$

$S1 := 0.333$

$S2 := 0.0001$

The output is as follows, with y2 the depth on the slope::

$y1 := 0.02ft$

$y2 := 0.25ft$ (Apron Depth)

Step 2

yc = 0.07 ft

Since the flow goes from supercritical to subcritical a hydraulic jump will occur.

Step 3

Need to manually iterate y3 until F1-F3 = M3-M1.

Channel Characteristic

$y1 = 0.02\,ft$

$B = 6\,ft \qquad Q = 0.71\,\frac{ft^3}{s}$

$Z = 3$

$A1 := y1 \cdot (B + Z \cdot y1)$

$A1 = 0.121\,ft^2$

$y3 := 0.1927\,ft$

$A3 := y3 \cdot (B + Z \cdot y3)$

$A3 = 1.268\,ft^2$

Depth to Centroid

$$z1 := \frac{\left(y1^3 + \frac{y1}{2} \cdot B \cdot y1\right)}{\left(y1^2 \cdot Z + B \cdot y1\right)}$$

$z1 = 9.967 \times 10^{-3}\,ft$

$$z3 := \frac{\left(\frac{B \cdot y3 \cdot y3}{2} + y3^3\right)}{\left(y3^2 \cdot Z + B \cdot y3\right)}$$

$z3 = 0.094\,ft$

Momentum

$F1 := z1 \cdot A1$

$F1 = 1.208 \times 10^{-3}\,ft^3$

$F3 := z3 \cdot A3$

$F3 = 0.119\,ft^3$

$g := 32.2\,\frac{ft}{sec^2}$

$$M3 := \frac{Q^2}{[g \cdot (B + Z \cdot y3) \cdot y3]}$$

$M3 = 0.012ft^3$

$$M1 := \frac{Q^2}{[g \cdot (B + Z \cdot y1) \cdot y1]}$$

$M1 = 0.129ft^3$

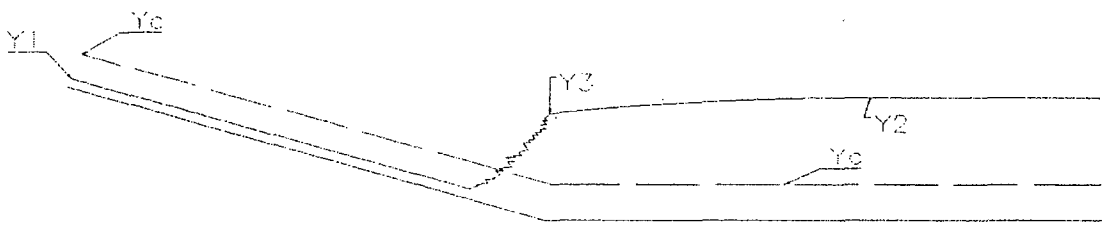
$M3 - M1 = -0.117ft^3$

$F1 - F3 = -0.117ft^3$

Step 4

The normal depth of the apron is 0.25 feet. This is more than the conjugate depth for the spillway. Therefore an S1 water surface profile will occur and the jump occurs upstream of the slope transition. Because of this the jump will not occur on the rip-rap.

Below is an image depicting the M3 water surface profile hydraulic jump.



Step 5

Since the jump has an S1 profile there is no need to calculate the length of the jump. Therefore this step was skipped.

7.3 Erosion in Rip-Rap Lined Channel

See attached spreadsheet calculation. Velocity on the apron is 0.42 fps and riprap is designed for at least 9 fps, therefore, erosion in the riprap lined channel will not occur while passing the PMP.

D, ft = (6*W50/(pi * gamma stone))^0.33

USACOE Hydraulic Design Criteria, Sheet 712-1. revised 9/1970
Stone Stability
Velocity vs. Stone diameter

6
3.141593

V = c*(2g*(gs-gs/gw)^0.5*D50^0.5

D50, ft = (6*W50/(pi * gamma stone))^0.33

Gamma Stone (gs)	165	pcf
g	32.2	ft/sec/sec
c, 0.86 high turb, 1.2 low	0.86	
gamma water (gw)	62.4	pcf

Bell Bend Rip Rap D50 ~ 100 lbs

D50, ft	V
1.05	9.07 fps

Stone Wt lbs	Eq Diam. ft
10	0.49
20	0.61
30	0.70
40	0.77
50	0.83
60	0.89
70	0.93
80	0.97
90	1.01
100	1.05
110	1.08
120	1.12
130	1.15
140	1.17
150	1.20
160	1.23
170	1.25

ESWEMS Pond Spillway Design
Calculation 161642.51.1512, Rev. 0

Appendix 1

Design Input References for Hydraulic Calculations

B&V Drawing 161642-1EMS-S1102
Properties of Areas
Geometric Elements of Channel Sections
Manning's N
Hydraulic Jump Shapes M3 and S1
Example Calculation, Hydraulic Jump Location

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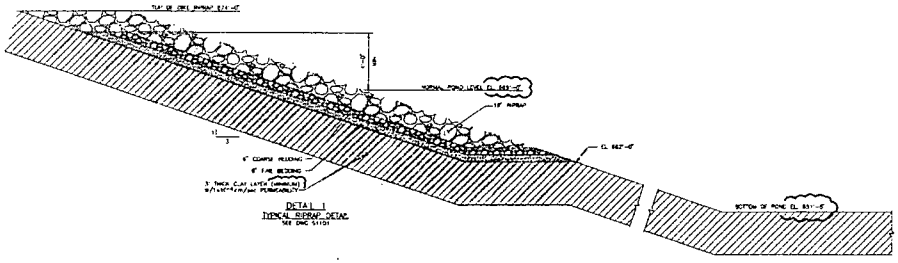


DETAIL 1
TYPICAL RIPRAP DETAIL
SEE DWG S1101

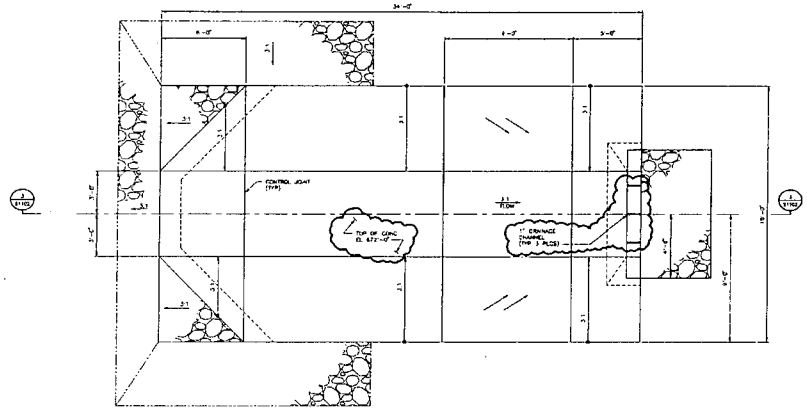
RIPRAP	
WEIGHT (lbs)	% SMALLER BY WEIGHT
500	100
350	92-98
225	75-85
100	45-55
12	8-12

COARSE BEDDING	
SIEVE SIZE*	% FINER BY WEIGHT
2"	100
1-1/2"	95-100
3/4"	35-70
3/8"	10-30
#4	0-5

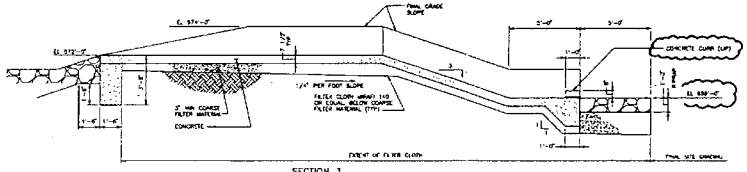
FINE BEDDING	
SIEVE SIZE*	% FINER BY WEIGHT
3/8"	100
#4	95-100
#8	80-100
#16	50-85
#30	25-60
#50	5-30
#100	0-10

*U.S. BUREAU OF STANDARDS
—ASTM C-33, FINE CONCRETE AGGREGATE

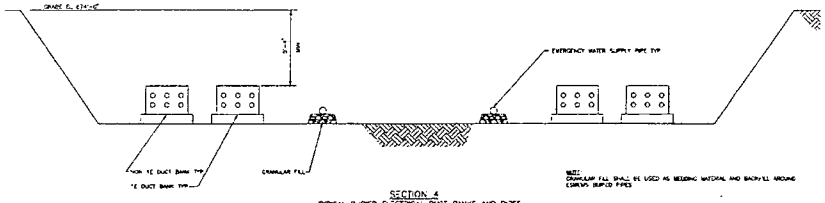
RIPRAP GRADATION



DETAIL 2
SPILLWAY PLAN
SEE DWG S1102



SECTION 3
SPILLWAY SECTION
SEE BHS DWG



SECTION 4
TYPICAL BURIED ELECTRICAL DUCT BANKS AND PIPES
SEE DWG S1101

NOTE:
GRANULAR FILL SHALL BE USED AS BEDDING MATERIAL AND BACKFILL AROUND EMBEDDED PIPES

NOTES

SEE DRAWING S1000 FOR GENERAL NOTES

NOT TO BE USED
FOR CONSTRUCTION

REV	DATE	DESCRIPTION	BY
1	04/30/2009	GENERAL REVISION	BARNES/SUNDLOV
0	10/29/2008	ISSUED FOR USE NAR-0013	ABELINE/SUNDLOV

THIS DOCUMENT CONTAINS SEISMIC CATEGORY I ITEMS ☒ YES ☐ NO

THIS DOCUMENT CONTAINS SAFETY-RELATED ITEMS ☒ YES ☐ NO

BLACK & VEATCH CORPORATION

DRAWN: RG DATE: MED
CHECKED: DATE:

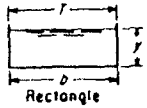
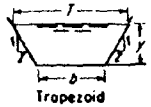
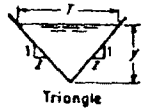
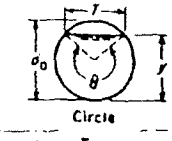
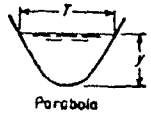
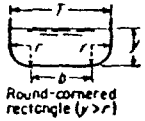
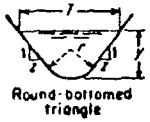
UNISTAR
BELL BEND NUCLEAR POWER PLANT

PLANT ARRANGEMENT ESWEMS POND
SECTIONS AND DETAILS

PROJECT DRAWING NUMBER: 161642-1EMS-S1102 REV: 1

Calc. 161642.
S1,S12, R/0
Appendix 1
Page 2 of 8

Table 2-1. Geometric Elements of Channel Sections

Section	Area A	Wetted perimeter P	Hydraulic radius R	Top width T	Hydraulic depth D	Section factor Z
 Rectangle	by	$b + 2y$	$\frac{by}{b + 2y}$	b	y	$by^{1.5}$
 Trapezoid	$(b + zy)y$	$b + 2y\sqrt{1 + z^2}$	$\frac{(b + zy)y}{b + 2y\sqrt{1 + z^2}}$	$b + 2zy$	$\frac{(b + zy)y}{b + 2zy}$	$\frac{[(b + zy)y]^{1.5}}{\sqrt{b + 2zy}}$
 Triangle	zy^2	$2y\sqrt{1 + z^2}$	$\frac{zy}{2\sqrt{1 + z^2}}$	$2zy$	$\frac{1}{2}y$	$\frac{\sqrt{2}}{2} zy^{2.5}$
 Circle	$\frac{1}{8}(\theta - \sin\theta)d_0^2$ 1/8	$\frac{1}{2}\theta d_0$	$\frac{1}{4}\left(1 - \frac{\sin\theta}{\theta}\right) d_0$	$(\sin\frac{1}{2}\theta)d_0$ or $2\sqrt{y(d_0 - y)}$	$\frac{1}{8}\left(\frac{\theta - \sin\theta}{\sin\frac{1}{2}\theta}\right) d_0$	$\frac{\sqrt{2}}{32} \frac{(\theta - \sin\theta)^{1.5}}{(\sin\frac{1}{2}\theta)^{0.5}} d_0^{2.5}$
 Parabola	$\frac{2}{3}Ty$	$T + \frac{8}{3}\frac{y^2}{T}$ *	$\frac{2T^2y}{3T^2 + 8y^2}$ *	$\frac{3}{2}\frac{A}{y}$	$\frac{2}{3}y$	$\frac{2}{3}\sqrt{6}\, Ty^{1.5}$
 Round-cornered rectangle ($y > r$)	$\left(\frac{\pi}{2} - 2\right) r^2 + (b + 2r)y$	$(\pi - 2)r + b + 2y$	$\frac{(\pi/2 - 2)r^2 + (b + 2r)y}{(\pi - 2)r + b + 2y}$	$b + 2r$	$\frac{(\pi/2 - 2)r^2}{b + 2r} + y$	$\frac{[(\pi/2 - 2)r^2 + (b + 2r)y]^{1.5}}{\sqrt{b + 2r}}$
 Round-bottomed triangle	$\frac{T^2}{4z} - \frac{r^2}{z}(1 - z\cot^{-1} z)$	$\frac{T}{z}\sqrt{1 + z^2} - \frac{2r}{z}(1 - z\cot^{-1} z)$	$\frac{A}{P}$	$2[z(y - r) + r\sqrt{1 + z^2}]$	$\frac{A}{T}$	$A\sqrt{\frac{A}{T}}$

* Satisfactory approximation for the interval $0 < x \leq 1$, where $x = 4y/T$. When $x > 1$, use the exact expression $P = (T/2)[\sqrt{1 + x^2} + 1/x \ln(x + \sqrt{1 + x^2})]$.

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R/O Appendix 1
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where 1.486 is the cube root of 3.28, the number of feet [illegible] pite the dimensional difficulties of the Manning formula, which have long plag[illegible]d those attempting to put all fluid mechanics on a rational dimensionless basis, it continues to be popular because it is simple to use and reasonably accurate. Representative values of n for various surfaces are given in Table 10.1.

In terms of flow rate, Eqs. (10.7) may be expressed as

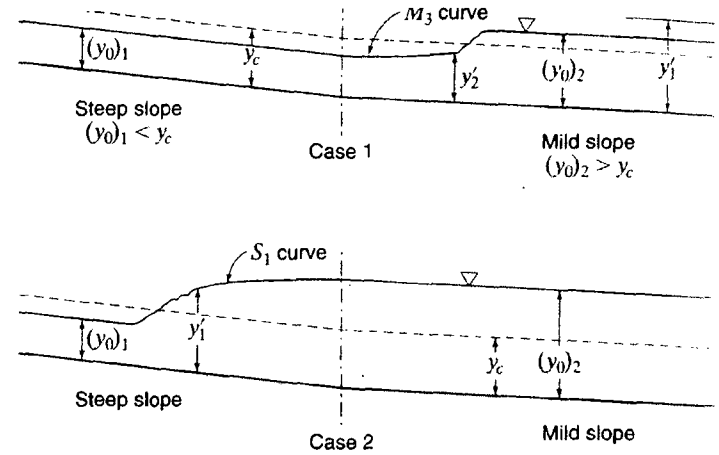
In BG units: $$Q(\text{cfs}) = \frac{1.486}{n} A R_h^{2/3} S_0^{1/2} \tag{10.8a}$$

In SI units: $$Q(\text{m}^3/\text{s}) = \frac{1}{n} A R_h^{2/3} S_0^{1/2} \tag{10.8b}$$

Table 10.1 Values of n in Manning's formula

Nature of surface	n Min	n Max
Lucite	0.008	0.010
Glass	0.009	0.013
Neat cement surface	0.010	0.013
Wood-stave pipe	0.010	0.013
Plank flumes, planed	0.010	0.014
Vitrified sewer pipe	0.010	0.017
Concrete, precast	0.011	0.013
Metal flumes, smooth	0.011	0.015
Cement mortar surfaces	0.011	0.015
Plank flumes, unplaned	0.011	0.015
Common-clay drainage tile	0.011	0.017
Concrete, monolithic	0.012	0.016
Brick with cement mortar	0.012	0.017
Cast iron, new	0.013	0.017
Riveted steel	0.017	0.020
Cement rubble surfaces	0.017	0.030
Canals and ditches, smooth earth	0.017	0.025
Corrugated metal pipe	0.021	0.030
Metal flumes, corrugated	0.022	0.030
Canals		
Dredged in earth, smooth	0.025	0.033
In rock cuts, smooth	0.025	0.035
Rough beds and weeds on sides	0.025	0.040
Rock cuts, jagged and irregular	0.035	0.045
Natural streams		
Smoothest	0.025	0.033
Roughest	0.045	0.060
Very weedy	0.075	0.150

Calc. 161642.51.1512, R/0
Appendix 1, page 4/8



e
draulic
e
ı is
y_1' is the
ı of $(y_0)_1$
ıjugate

Sample Problem 10.10 Analyze the water-surface profile in a long rectangular channel lined with concrete ($n = 0.013$). The channel is 10 ft wide, the flow rate is 400 cfs, and the channel slope changes abruptly from 0.0150 to 0.0016. Find also the horsepower loss in the resulting jump.

Solution

Eq. (10.8*a*):
$$400 = \frac{1.486}{0.013}(10y_{0_1})\left(\frac{10y_{0_1}}{10 + 2y_{0_1}}\right)^{2/3}(0.015)^{1/2}$$

We can solve this equation by any of the methods described in Sample Prob. 10.1. We obtain

$$y_{0_1} = 2.17 \text{ ft} \qquad \text{(normal depth on the upper slope)}$$

Using a similar procedure, we find the normal depth y_{0_2} on the lower slope to be 4.81 ft.

Eq. (10.23):
$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left[\frac{\left(\frac{400}{10}\right)^2}{32.2}\right]^{1/3} = 3.68 \text{ ft}$$

Thus flow is supercritical ($y_{0_1} < y_c$) before the break in slope and subcritical ($y_{0_2} > y_c$) after the break, so a hydraulic jump must occur. One of the two profiles (case 1 or case 2) in Fig. 10.26 must occur.

Let us first explore case 2. Applying Eq. (10.46*a*) to determine the depth conjugate to the 2.17-ft normal depth on the upper slope, we get

$$y_1' = \frac{2.17}{2}\left\{-1 + \left[1 + \frac{8(40)^2}{32.2(2.17)^3}\right]^{1/2}\right\} = 5.77 \text{ ft}$$

: Programmed computing aids (Appendix C) could help solve problems marked with this icon.

Calc. 161642.51, 1512, R10
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Therefore a jump on the upper slope must rise to 5.77 ft, must rise still more along an S_1 curve (Fig. 10.20). When the slope, the depth would therefore be greater than $y_{0_2} = 4.8$ Because an M_1 curve (Fig. 10.20) cannot bring the wate above 5.77 ft to 4.81 ft (normal depth), such a profile and ji

Let us now explore case 1. Applying Eq. (10.46*b*) to conjugate to the 4.81-ft normal depth on the lower slope,

$$y_2' = \frac{4.81}{2}\left\{-1 + \left[1 + \frac{8(40)^2}{32.2(4.81)^3}\right]^{1/2}\right\} =$$

This lower conjugate depth of 2.74 ft will occur downst slope. The water surface on the lower slope *can* rise from via an M_3 curve (Fig. 10.20). So this case will occur.

We can find the location of the jump (i.e., its distan slope) by applying Eq. (10.39) to the M_3 curve between break and 2.74 ft at the start of the jump:

$$\Delta x = \frac{E_1 - E_2}{S - S_0}$$

$$E_1 = 2.17 + \frac{(40/2.17)^2}{2(32.2)} = 7.45 \text{ ft}$$

$$E_2 = 2.74 + \frac{(40/2.74)^2}{2(32.2)} = 6.05 \text{ ft}$$

$$\bar{V} = \frac{1}{2}\left(\frac{40}{2.17} + \frac{40}{2.74}\right) = 16.53 \text{ fps}$$

$$\bar{R}_h = \frac{1}{2}\left(\frac{21.7}{14.34} + \frac{27.4}{15.47}\right) = 1.641 \text{ ft}$$

From Eq. (10.38*a*): $$S = \left[\frac{(0.013)(16.53)}{1.486(1.641)^{2/3}}\right]^2 = 0.01081$$

Eq. (10.39): $$\Delta x = \frac{7.452 - 6.054}{0.01081 - 0.00160} = 151.8 \text{ ft}$$

Note that we could compute this distance more accurate more reaches, i.e., by using more, smaller depth increment

Thus the depth on the upper slope is 2.17 ft; downstr depth increases gradually (M_3 curve) to 2.74 ft over a dista 152 ft; then a hydraulic jump occurs from a depth of 2.74 ft t of the jump the depth remains constant (i.e., normal) at 4.81

Eq. (10.48): $$h_{L_j} = \frac{(4.81 - 2.74)^3}{4(4.81)2.74} = 0.1695 \text{ ft}$$

So $$P \text{ loss} = \frac{\gamma Q h_{L_j}}{550} = \frac{62.4(400)0.1695}{550} = 7.69 \text{ hp}$$

Calc. 16/642.51.1512, R/a
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(BG units)

In free-surface flow such as this where the streamlin water surface is coincident with the hydraulic grade line. equation from the upstream section to the downstream sec

$$6 + \frac{V_1^2}{2g} = 3 + \frac{V_2^2}{2g}$$

From continuity, $$6(10)V_1 = 3(10)V_2$$

Substituting Eq. (2) into Eq. (1) yields

$$V_1 = 8.02 \text{ fps}, \quad V_2 = 16.05 \text{ fps}$$

$$Q = A_1V_1 = A_2V_2 = 481 \text{ cfs}$$

Next take a free-body diagram of the control volume (*CV*) the figure and apply the momentum equation (6.7*a*),

$$F_1 - F_2 - F_x = \rho Q(V_2 - V_1)$$

where F_x represents the force of the structure on the water (C tal direction, and the *F*'s and *V*'s are understood to have no *y*

From Eq. (3.16), we have $F_1 = \gamma h_{c1}A_1$ and $F_2 = \gamma h_{c2}A_2$. I

$$62.4(3)(10 \times 6) - 62.4(1.5)(10 \times 3) - F_x = 1.94(481)(1$$

and $$F_x = +936 \text{ lb} = 936 \text{ lb} \leftarrow$$

The positive sign means that the assumed direction is correc water on the structure is equal and opposite, namely,

$$(F_{W/S})_x = 936 \text{ lb} \rightarrow \qquad \textbf{ANS}$$

Note that the momentum principle will not permit us to component of the force of the water on the shaded struc pressure distribution along the bottom of the channel is u estimate the pressure distribution along the boundary of the st the bottom of the channel by sketching a flow net and ap principle. Then we can find the horizontal and vertical compo by computing the integrated effect of the pressure-distributio

(SI units)

Energy: $$2 + \frac{V_1^2}{2(9.81)} = 1 + \frac{V_2^2}{2(9.81)}$$

Continuity: $$2(3)V_1 = 1(3)V_2$$

Substituting Eq. (4) into Eq. (3) yields

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42 Properties of Areas

Nomenclature

A	area	units^2
	base distance	units
	distance to extreme fiber	units
	separation distance	units
	height distance	units
	moment of inertia	units^4
J	polar moment of inertia	units^4
L	length	units
P	product of inertia	units^4
Q	first moment of the area	units^3
	radius	units
	radius of gyration	units
S	section modulus	units^3
V	volume	units^3
x	distance in the x-direction	units
y	distance in the y-direction	units

Symbols

θ	angle	rad

Subscripts

o	with respect to the origin
c	centroidal

1. CENTROID OF AN AREA

The *centroid* of an area is analogous to the center of gravity of a homogeneous body.[1] The centroid is often described as the point at which a thin homogeneous plate would balance. This definition, however, combines the definitions of centroid and center of gravity and implies that gravity is required to identify the centroid, which is not true.

[1] The analogy has been simplified. A three-dimensional body also has a centroid. However, the centroid and center of gravity will be coincide unless the body is homogeneous.

The location of the centroid of an area bounded by the x- and y-axes and the mathematical function $y = f(x)$ can be found by the *integration method* by using Eqs. 42.1 through 42.4. The centroidal location depends only on the geometry of the area and is identified by the coordinates (x_c, y_c). Some references place a bar over the coordinates of the centroid to indicate an average point, such as $(\bar{x}, \bar{y})$.

$$x_c = \frac{\int x\,dA}{A} \qquad 42.1$$

$$y_c = \frac{\int y\,dA}{A} \qquad 42.2$$

$$A = \int f(x)\,dx \qquad 42.3$$

$$dA = f(x)\,dx = g(y)\,dy \qquad 42.4$$

The locations of the centroids of *basic shapes*, such as triangles and rectangles, are well known. The most common basic shapes have been included in App. 42.A. There should be no need to derive centroidal locations for these shapes by the integration method.

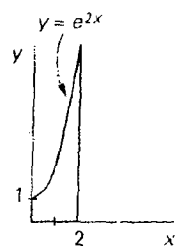
The centroid of a complex area can be found from Eqs. 42.5 and 42.6 if the area can be divided into the basic shapes in App. 42.A. This process is simplified when all or most of the subareas adjoin the reference axis. Example 42.1 illustrates this method.

$$x_c = \frac{\sum_i A_i x_{ci}}{\sum_i A_i} \qquad 42.5$$

$$y_c = \frac{\sum_i A_i y_{ci}}{\sum_i A_i} \qquad 42.6$$

Example 42.1

An area is bounded by the x- and y-axes, the line $x = 2$, and the function $y = e^{2x}$. Find the x-component of the centroid.



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Structural

ESWEMS Pond Spillway Design
Calculation 161642.51.1512, Rev. 0

Appendix 2

Bently Flowmaster Calculations
Hand Calculation verifying Flowmaster Results

Spillway

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.33330	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	6.00	ft
Discharge	0.71	ft^3/s

Results

Normal Depth	0.02	ft
Flow Area	0.14	ft^2
Wetted Perimeter	6.14	ft
Top Width	6.13	ft
Critical Depth	0.07	ft
Critical Slope	0.00594	ft/ft
Velocity	5.23	ft/s
Velocity Head	0.43	ft
Specific Energy	0.45	ft
Froude Number	6.20	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.02	ft
Critical Depth	0.07	ft
Channel Slope	0.33330	ft/ft
Critical Slope	0.00594	ft/ft

Calc. 161642.S1.1512, K/6
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Apron

Project Description

Friction Method	Manning Formula	
Solve For	Normal Depth	

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.00010	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	6.00	ft
Discharge	0.71	ft^3/s

Results

Normal Depth	0.25	ft
Flow Area	1.69	ft^2
Wetted Perimeter	7.58	ft
Top Width	7.50	ft
Critical Depth	0.07	ft
Critical Slope	0.00594	ft/ft
Velocity	0.42	ft/s
Velocity Head	0.00	ft
Specific Energy	0.25	ft
Froude Number	0.16	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.25	ft
Critical Depth	0.07	ft
Channel Slope	0.00010	ft/ft
Critical Slope	0.00594	ft/ft

Calc. 161642.51.1512, R/0
Appendix 2, page 3/4

BLACK & VEATCH				
	OWNER	Unistar	COMP'D BY	SJW
	PLANT	Bell Bend Nuclear	DATE	6/26/2009
	PROJECT NO.	161642	CKD BY	R. Gorny
	TITLE	Bentley FlowMaster Verification	DATE	July 1, '09
	File No.		PAGE	
			Template Rev.	0

Manning's Equation:

Q = 1.49/n*A*(A/P)^(2/3)*S^0.5

Where:

n=	manning's roughness coefficient
y=	normal depth, ft
b=	bottom width, ft
z=	side slope
A=	cross-sectional area, ft^2
	A = b*y + z*y^2
P=	wetted perimeter, ft
	P= b+((1+z)^0.5)*2*y
S=	slope, ft/ft
Q=	flow in channel, cfs

Critical Depth Equation

Q^2/g=A^3/B

Where:

Q=	flow in channel, cfs
g=	gravity constant
A=	cross-sectional area, ft^2
	$A := by_c + z(y_c)^2$
B=	top width, ft
	$B := b + 2zy_c$

Normal Depth Calculation

y=	0.02 ft*
n=	0.013
S=	0.3330 ft/ft
A=	0.14 ft^2
P=	6.14 ft
z=	3 ft/ft
b=	6 ft
Q=	0.7 cfs
V=	5.22 ft/s

@ slope portion comparing to page 2 of the appendix, V = 5.23 ft/s ~ O.K.

Critical Depth Calculation

yc=	0.80 ft*
Q=	30.00 cfs
g=	32.2 ft/s^2
A^3=	301.656 ft^6
B=	10.79 ft

* The goal seek function was used to determine the depth.

Calc 161642.51.1512
R/0, Appendix 2, page 4/4

ESWEMS Pond Spillway Design
Calculation 161642.51.1512, Rev. 0

Appendix 3

USACE Hydraulic Design Criteria
Sheet 712-1
Stone Stability
Velocity vs Stone Diameter

HYDRAULIC DESIGN CRITERIA

SHEET 712-1

STONE STABILITY

VELOCITY VS STONE DIAMETER

1. Purpose. Hydraulic Design Chart 712-1 can be used as a guide for the selection of rock sizes for riprap for channel bottom and side slopes downstream from stilling basins and for rock sizes for river closures. Recommended stone gradation for stilling basin riprap is given in paragraph 6.

2. Background. In 1885 Wilfred Airy[1] showed that the capacity of a stream to move material along its bed by sliding is a function of the sixth power of the velocity of the water.[1] Henry Law applied this concept to the overturning of a cube,[2] and in 1896 Hooker[2] illustrated its application to spheres. In 1932 and 1936 Isbash published coefficients for the stability of rounded stones dropped in flowing water.[3,4] The design curves given in Chart 712-1 have been computed using Airy's law and the experimental coefficients for rounded stones published by Isbash.

3. Theory. According to Isbash the basic equation for the movement of stone in flowing water can be written as:

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}(D)^{1/2} \qquad (1)$$

where

V = velocity, fps

C = a coefficient

g = acceleration of gravity, ft/sec^2

γ_s = specific weight of stone, lb/ft^3

γ_w = specific weight of water, lb/ft^3

D = stone diameter, ft

The diameter of a spherical stone in terms of its weight W is

$$D = \left(\frac{6W}{\pi\gamma_s}\right)^{1/3} \qquad (2)$$

Substituting for D in equation 1 results in

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}\left(\frac{6W}{\pi\gamma_s}\right)^{1/6} \qquad (3)$$

which describes Airy's law stated in paragraph 2.

4. Experimental Results. Experimental data on stone movement in flowing water from the early (1786) work of DuBuat[5] to the more recent Bonneville Hydraulic Laboratory tests[6] have been shown to confirm Airy's law and Isbash's stability coefficients.[7] The published experimental data are generally defined in terms of bottom velocities. However, some are in terms of average flow velocities and some are not specified. The Isbash coefficients are from tests with essentially no boundary layer development and the average flow velocities are representative of the velocity against stone. When the stone movement resulted by sliding, a coefficient of 0.86 was obtained. When movement was effected by rolling or overturning, a coefficient of 1.20 resulted. Extensive U. S. Army Engineer Waterways Experiment Station laboratory testing for the design of riprap below stilling basins indicates that the coefficient of 0.86 should be used with the average flow velocity over the end sill for sizing stilling basin riprap because of the excessively high turbulence level in the flow. For impact-type stilling basins, the Bureau of Reclamation[8] has adopted a riprap design curve based on field and laboratory experience and on a study by Mavis and Laushey.[9] The Bureau curve specifies rock weighing 165 lb/ft^3 and is very close to the Isbash curve for similar rock using a stability coefficient of 0.86.

5. Application. The curves given in Chart 712-1 are applicable to specific stone weights of 135 to 205 lb/ft^3. The use of the average flow velocity is desirable for conservative design. The solid-line curves are recommended for stilling basin riprap design and other high-level turbulence conditions. The dashed line curves are recommended for river closures and similar low-level turbulence conditions. Riprap bank and bed protection in natural and artificial flood-control channels should be designed in accordance with reference 10.

6. Stilling Basin Riprap.

a. Size. The W_{50} stone weight and the D_{50} stone diameter for establishing riprap size for stilling basins can be obtained using Chart 712-1 in the manner indicated by the heavy arrows thereon. The effect of specific weight of the rock on the required size is indicated by the vertical spread of the solid line curves.

b. Gradation. The following size criteria should serve as guidelines for stilling basin riprap gradation.

(1) The lower limit of W_{50} stone should not be less than the weight of stone determined using the appropriate "Stilling Basins" curve in Chart 712-1.

(2) The upper limit of W_{50} stone should not exceed the weight that can be obtained economically from the quarry or the size that will satisfy layer thickness requirements as specified in paragraph 6c.

(3) The lower limit of W_{100} stone should not be less than two times the lower limit of W_{50} stone.

(4) The upper limit of W_{100} stone should not be more than five times the lower limit of W_{50} stone, nor exceed the size that can be obtained economically from the quarry, nor exceed the size that will satisfy layer thickness requirements as specified in paragraph 6c.

(5) The lower limit of W_{15} stone should not be less than one-sixteenth the upper limit of W_{100} stone.

(6) The upper limit of W_{15} stone should be less than the upper limit of W_{50} stone as required to satisfy criteria for graded stone filters specified in EM 1110-2-1901.

(7) The bulk volume of stone lighter than the W_{15} stone should not exceed the volume of voids in the revetment without this lighter stone.

(8) W_0 to W_{25} stone may be used instead of W_{15} stone in criteria (5), (6), and (7) if desirable to better utilize available stone sizes.

c. Thickness. The thickness of the riprap protection should be $2D_{50\ max}$ or $1.5D_{100\ max}$, whichever results in the greater thickness.

d. Extent. Riprap protection should extend downstream to where nonerosive channel velocities are established and should be placed sufficiently high on the adjacent bank to provide protection from wave wash during maximum discharge. The required riprap thickness is determined by substituting values for these relations in equation 2.

7. References.

(1) Shelford, W., "On rivers flowing into tideless seas, illustrated by the river Tiber." Proceedings, Institute of Civil Engineers, vol 82 (1885).

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(3) Isbash, S. V., Construction of Dams by Dumping Stones in Flowing

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(4) _____, "Construction of dams by depositing rock in running water." Transactions, Second Congress on Large Dams, vol 5 (1936), pp 123-136.

(5) DuBuat, P. L. G., Traite d'Hydraulique. Paris, France, 1786.

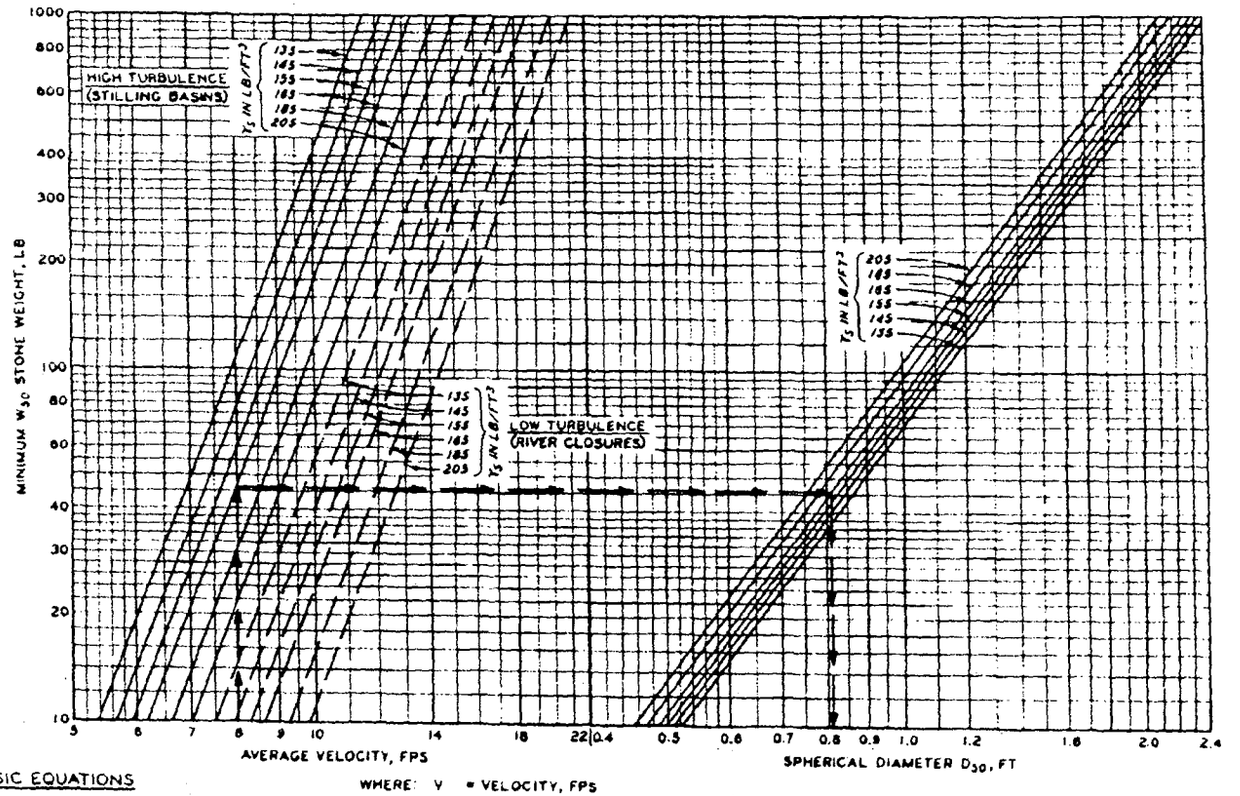
(6) U. S. Army Engineer District, Portland, CE, McNary Dam - Second Step Cofferdam Closure. Bonneville Hydraulic Laboratory Report No. 51-1, 1956.

(7) U. S. Army Engineer Waterways Experiment Station, CE, Velocity Forces on Submerged Rocks. Miscellaneous Paper No. 2-265, Vicksburg, Miss., April 1958.

(8) U. S. Bureau of Reclamation, Stilling Basin Performance; An Aid in Determining Riprap Sizes, by A. J. Peterka. Hydraulic Laboratory Report No. HYD-409, Denver, Colo., 1956.

(9) Mavis, F. T. and Laushey, L. M., "A reappraisal of the beginning of bed movement - competent velocity." Second Meeting, International Association for Hydraulic Structure Research, Stockholm, Sweden, 1948. See also Civil Engineering, vol 19 (January 1949), pp 38, 39, and 72.

(10) U. S. Army, Office, Chief of Engineers, Engineering and Design; Hydraulic Design of Flood Control Channels. EM 1110-2-1601, Washington, D. C., 1 July 1970.



BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}(D_{50})^{1/2}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi\gamma_s}\right)^{1/3}$$

WHERE:
- V = VELOCITY, FPS
- γ_s = SPECIFIC STONE WEIGHT, LB/FT³
- γ_w = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT³
- W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.
- D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}
- C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)
- g = ACCELERATION OF GRAVITY, FT/SEC²

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 1 OF 2)

REV 8-58, 9-70 WES 6-57

Calc. 16/642.51, 1512
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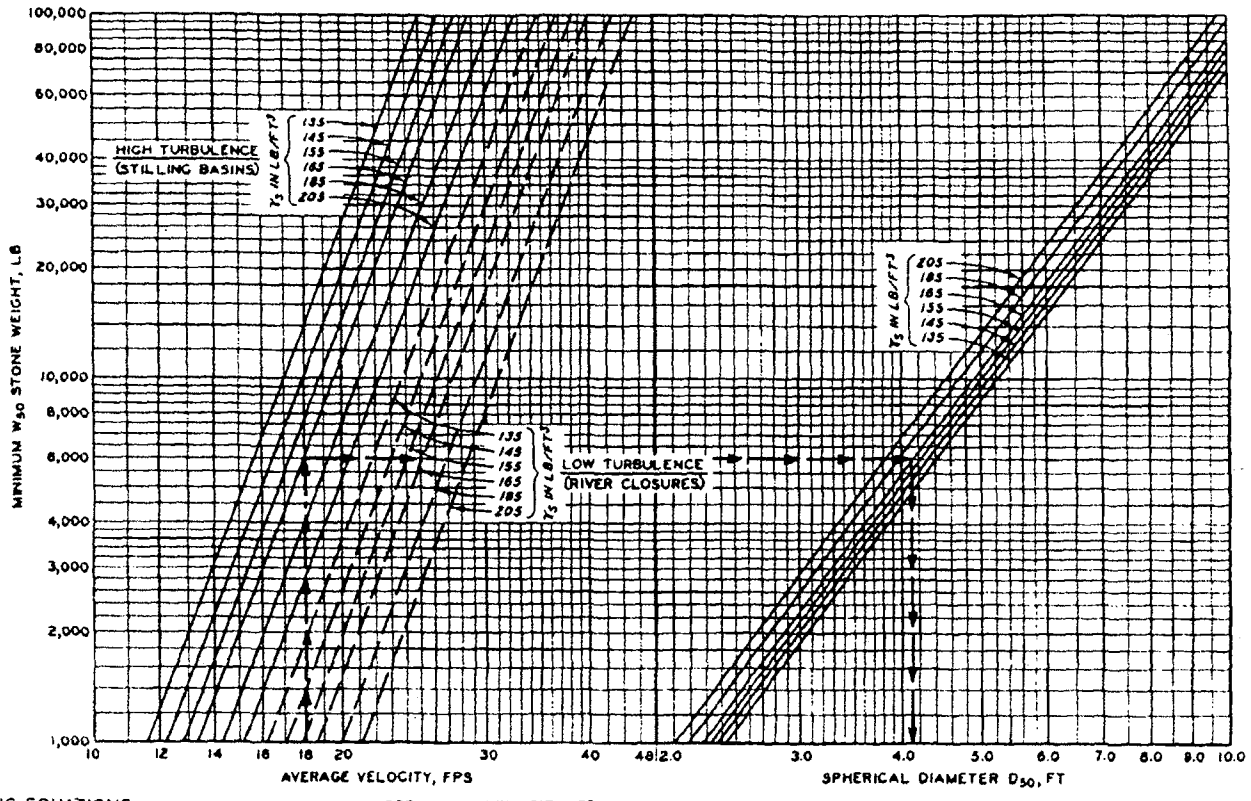

BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}(D_{50})^{1/2}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi\gamma_s}\right)^{1/3}$$

WHERE:
- V = VELOCITY, FPS
- γ_s = SPECIFIC STONE WEIGHT, LB/FT³
- γ_w = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT³
- W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.
- D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}
- C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)
- g = ACCELERATION OF GRAVITY, FT/SEC²

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 2 OF 2)

REV 8-58, 9-70 WES 8-57

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ESWEMS Pond Spillway Design
Calculation 161642.51.1512, Rev. 0

Attachment A

PCR Calculation
PMF for Bell Bend ESWEMS Pond

F-4 4d 29/44

Paul C. Rizzo Associates, Inc.
CONSULTANTS

By DJW Date 10/1/08 Subject PMF for Bell Bend NPP Sheet No. 1 of 64
Chkd. by BKF Date 10/8/08 ESWEMS Pond Proj. No. 07-3891

DJW = David Wallner

BKF = Ben Ferguson

1.0 PURPOSE

The purpose of this calculation is to determine the PMF water elevation in the ESWEMS Pond resulting from the 72-hour PMP event. Under the assumption that no losses occur, the 72-hour PMP event is the worst-case scenario when analyzing the ESWEMS Pond since it generates more total rainfall than the 1-hour PMP event, which was used as the basis for evaluating the entire BBNPP site drainage system (***Reference 12***). The 1-hour PMP event is the worst-case scenario in the sub-basins containing all safety-related structures (except the ESWEMS Pond) since peak discharges are higher due to the increased intensity of the rainfall, causing the water surface levels to rise higher during the runoff process.

2.0 REFERENCES

1. Wallner, David (August 5, 2008), "Probable Maximum Storm Event for Walker Run Watershed (BBNPP)," RIZZO Calculation, Project No. 07-3891.
2. Sargent & Lundy, "Conceptual Grading and Drainage Plan" Drawings, Drawings No. 12198-004-CSK-001 through 12198-004-CSK-014, Rev 7 (issued on August 11, 2008). (see **sheets 25 – 39**; these drawings were used as a reference in order to develop the site drawing that includes post-construction elevation contours, which is shown on **sheet 8**)
3. Natural Resource Conservation Service (June 1986), *Urban Hydrology for Small Watersheds (TR-55)*. (see **sheets 40 – 46** for excerpts)
4. US Army Corps of Engineers Hydrologic Engineering Center, *HEC-HMS 3.1.0.*
5. US Army Corps of Engineers Hydrologic Engineering Center (Mar 2000), *HEC-HMS Technical Reference Manual*. (see **sheets 47 – 50** for excerpts)
6. Baghdadi, Riad (June 2, 2008), Black & Veatch, *Bell Bend NPP Site Layout / Site Drainage Design*, e-mail. (correspondence and attached pdf file showing a schematic of the ESWEMS Pond design, see **sheets 51 – 54**)
7. Mesania, Fehmida (July 2, 2008), *RE: BBNPP UHS*, e-mail. (correspondence identifying a change in the ESWEMS Pond spillway elevation, see **sheets 55 – 56**)
8. Brater, Ernest F. and King, Horace W. (1976), *Handbook of Hydraulics: Sixth Edition*. (see **sheets 57 – 60** for excerpts)
9. AMEC Earth and Environmental (Aug 2001), *Georgia Stormwater Manual: Volume 2*. (see **sheets 61 – 63** for excerpts)
10. Mesania, Fehmida (2008), "Verification & Validation for HEC-HMS 3.1.0," RIZZO, 08-9006.
11. Ferguson, Ben (March 6, 2008), "PMF for Callaway NPP Unit 2 Site Drainage System," RIZZO Calculation, Project No. 06-3624. (ESWEMS Pond spillway length)
12. Wallner, David (September 29, 2008), "PMF for Bell Bend NPP Site Drainage System," RIZZO Calculation, Project No. 07-3891.

Calc. 16/642. 51. 1512, R/0
Attach. A, page 2/9

3.0 ASSUMPTIONS

1. Both SCS basin loss and hydrograph transformation methods are applied since the area is quite small (less than 10 acres).
2. The SCS Runoff Method parameters are as follows:
   a. The curve number (CN) is assumed to be 98.
   b. The initial abstraction is assumed to be 0 inches.
   c. The site is assumed to be 100% impervious.

   These parameters represent a worst case scenario in which all rainfall is converted to runoff.
3. The spillway at the ESWEMS Pond is assumed to be a 6-foot long broad-crested weir at El. 672. The discharge coefficient for this weir is assumed to be 2.65 (***Reference 8***). A 6-foot spillway was assumed because this is the size of a spillway at a NPP site with a similar size ESWEMS Pond (***Reference 11***).
4. Overflow of the dike can be modeled as a broad crested weir with a discharge coefficient of 2.63 (***Reference 8***). However, it is not acceptable for either the ESWEMS Pond pool elevation or the wave runup to exceed the dike at El. 674.
5. The ESWEMS Pond is 700 ft long and 400 ft wide at El. 674 with 3H:1V side slopes (***Reference 6***).
6. As a worst-case scenario, the make-up pump at the ESWEMS Pond is assumed to be unusable during this storm event.
7. The PMP is for a 10 square mile area at the location of the site and taken from ***Reference 1***. The 72-hour PMP storm event (as opposed to the 1-hour event) was used to evaluate the PMF water elevation in the ESWEMS Pond since there is more total rainfall, allowing the water surface level to rise higher under the assumption that there are no losses.
8. The 2-year 24-hour rainfall used to calculate the travel time (T_t) for each sub-basin is 3.0" (***Reference 3***).
9. All elevations are referenced to Mean Sea Level (MSL), unless otherwise noted.
10. The assumed starting Water Surface Elevation (WSE) is El. 669, which is the specified normal water elevation in the ESWEMS Retention Pond (***Reference 6***).

4.0 METHODOLOGY

HEC-HMS is software published by the US Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC). Version 3.1.0 will be used to model the effect of the Probable Maximum Precipitation (PMP) at the site. HEC-HMS is capable of taking a specified hyetograph and calculating the runoff that is generated, balancing the inflows and outflows of a reservoir according to various discharge parameters. The inclusion of such features such as spillway and dam top overflow, SCS methods, and fine resolution 1-minute output make this software ideal for this calculation.

Parameters needed for the HEC-HMS Model are:

- Specified hyetograph for the 72-hr PMP
- Drainage area of the ESWEMS Pond

Calc. 16/642, 51.1512, R/0
Attach. A, page 3/9

- SCS Loss Parameters
  - SCS Curve Number
  - Initial Abstraction
  - Percentage of drainage area that is impervious
- SCS Unit Hydrograph Transformation
  - Lag Time
- Elevation-storage curve for the ESWEMS Pond drainage area
- Assumed starting elevation
- Discharge structure lengths, elevations, and coefficients

4.1 Delineation of Drainage Areas

The delineations of the drainage areas that make up the BBNPP site can be seen in ***Attachment A***. Only the ESWEMS Pond sub-basin is considered in this analysis. The drainage area of the ESWEMS Pond is presented in ***Table 4.1-1***.

TABLE 4.1-1

SUB-BASIN	AREA (FT^2)	AREA (ACRE)	AREA (MI^2)
ESWEMS Pond	272,153.2	6.248	0.009762

4.2 Determination of Lag Time

Lag time needed to be determined for the ESWEMS Pond drainage area. This was accomplished using the method described in ***Reference 3***. The worksheets provided in this reference allow for the calculation of the time of concentration T_c. Once T_c is known, the lag time T_{lag} can be determined from Equation 6-10 of ***Reference 5***:

$$T_{lag} = 0.6T_c$$

The calculations for lag time are attached as ***Attachment B***. The results for the ESWEMS Pond drainage area are presented in ***Table 4.2-1***.

TABLE 4.2-1

SUB-BASIN	T_C (HR)	T_C (MIN)	T_{LAG} (MIN)
ESWEMS Pond	0.00	1	1

4.3 Elevation-Storage Curve

The ESWEMS Pond used an elevation-storage curve that reflected a trapezoidal reservoir with the following parameters:

- Bottom elevation = 651.5 ft

Calc. 16/642, 51.1512, R/0
Attach. A, page 4/9

- Top elevation = 674 ft
- The dimensions of the reservoir top elevation (674 ft) are 700 ft long and 400 ft wide
- All sides have a slope of 3H:1V

With this information, Equation 2.2.3 of ***Reference 9*** can be used to generate the elevation-storage curve:

$$V = LWD + (L+W)ZD^2 + \frac{4}{3}Z^2D^3$$

In the above equation, Z is the side-slope factor and is equivalent to the ratio of the horizontal to the vertical ($Z = 3$). The resulting elevation-storage curve can be seen in ***Attachment C***.

The assumed starting Water Surface Elevation (WSE) is shown in ***Table 4.3-1***.

TABLE 4.3-1

RESERVOIR ELEMENT	STARTING WSE
ESWEMS Pond	669.0

4.4 Discharge Structures

ESWEMS Pond discharge structures are labeled as Spillway 1 and Dam Top 1 according to their names in the HEC-HMS model.

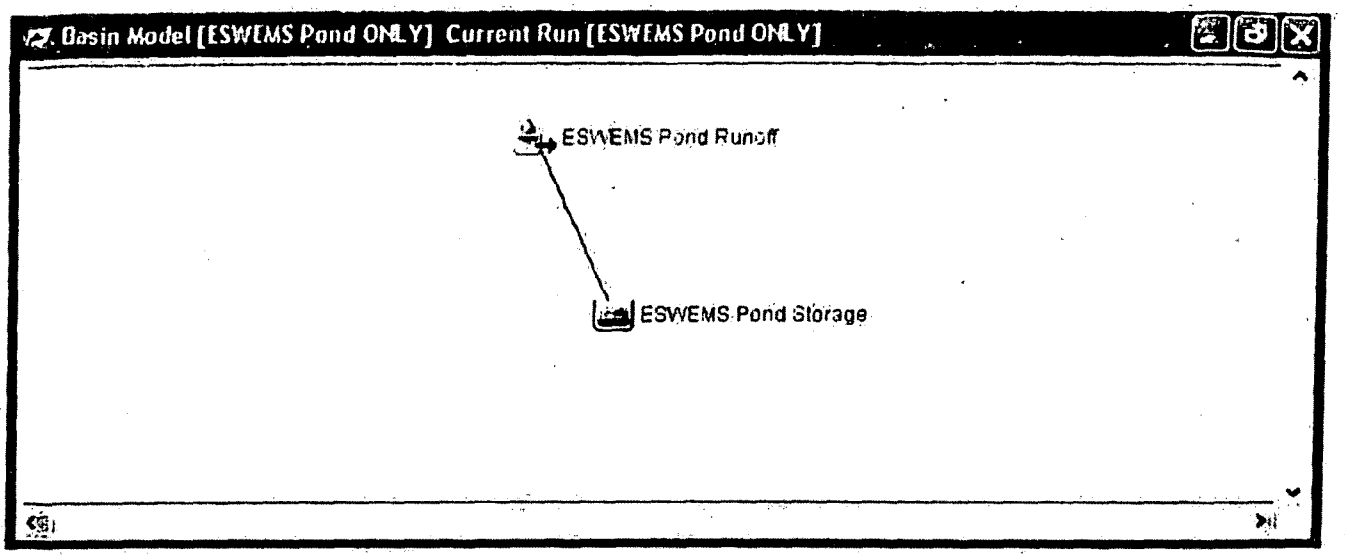
- **ESWEMS Pond**
  - Spillway 1
    - Flow over broad-crested weir
    - El. = 672
    - L = 6 ft
    - C = 2.65
    - *Note: The peak WSE in the ESWEMS Pond during the PMP storm event slightly exceeds the crest of Spillway 1, allowing runoff to discharge at a peak rate of 0.71 cfs into the surrounding "Switchyard" sub-basin. This issue must be addressed by Black and Veatch since the spillway crest elevation was designed NOT to be exceeded under these conditions (spillway elevation should be greater than the maximum water surface elevation resulting from the PMF).*
  - Dam Top 1
    - Dike overflow into Switchyard
    - El. = 674 (top of dike elevation)
    - L = 2,041.61 ft
    - C = 2.63
    - *Note: Dike is not overtopped in the analysis*

Calc. 161642.51.1512, R/0
Attach. A, page 5/9

4.5 HEC-HMS Model Setup

A Screenshot of the HEC-HMS model is shown below in ***Figures 4.5-1***.

FIGURE 4.5-1



Since the ESWEMS Pond is the only sub-basin that is modeled for this analysis, the HEC-HMS model is very basic: all runoff from the 72-hour PMP storm event is stored in the ESWEMS Pond, which has discharge structures as specified in the previous section.

4.6 LINKS TO ELECTRONIC FILES

- HEC-HMS files
  - G:\DJW\Berwick NPP (07-3891)\Model Based on Rev. 2 Drawings Dated 06-02-2008\BBNPP_Site_Drainage_PMF\BBNPP_Site
    - *Note: Although this is the HEC-HMS file that is based on the old Revision 2 S&L drawings that were issued on June 2, 2008, the design dimensions for the ESWEMS Pond were never changed in later revisions of the site layout. Therefore, since the model already contained the correct 72-hour PMP data, the "ESWEMS Pond ONLY" basin model was used for this analysis.*

- Proposed site grading CAD file with drainage area delineations
  - G:\DJW\Berwick NPP (07-3891)\Model Based on Rev. 7 Drawings Dated 08-11-2008\Drawings\Assumed Grade Contours (08-11-08 Drawings)

- Excel Files
  - G:\DJW\Berwick NPP (07-3891)\Model Based on Rev. 7 Drawings Dated 08-11-2008\Elevation-Storage Curves (08-11-08 Model)

**Do not modify these files. Please save a copy to your local hard drive before accessing.*

Calc 16164 2-51, 1512, R/0
Attach. A, page 6/9

5.0 RESULTS

The peak WSE for the ESWEMS Pond is presented in ***Table 5.0-1***.

TABLE 5.0-1

RESERVOIR ELEMENT	PEAK WSE (FT MSL)
ESWEMS Pond	672.13

Output from HEC-HMS is given in Appendix D.

6.0 CONCLUSIONS

The ESWEMS Pond should not overtop, and it is shown that this criterion is met after the 72-hour PMP storm event with 1.87 ft of freeboard (El. 674 – El. 672.13). However, a wind setup and wave runup analysis should be performed using this peak WSE to determine if overtopping from wave action is a possibility.

Since the crest of the spillway is exceeded during the 72-hour PMP event with a peak outflow over the spillway of 0.71 cfs, Black and Veatch should determine whether or not the ESWEMS Pond design is still acceptable since the spillway elevation should be greater than the maximum water surface elevation resulting from the PMF. Raising the spillway elevation will keep a small amount of water inside the pond, which will in turn cause a very slight increase in the water surface elevation.

ATTACHMENT A

ESWEMS Pond Drainage Area Delineation

Calc. 161642.51.1512, R10
Attach. A, page 8/9

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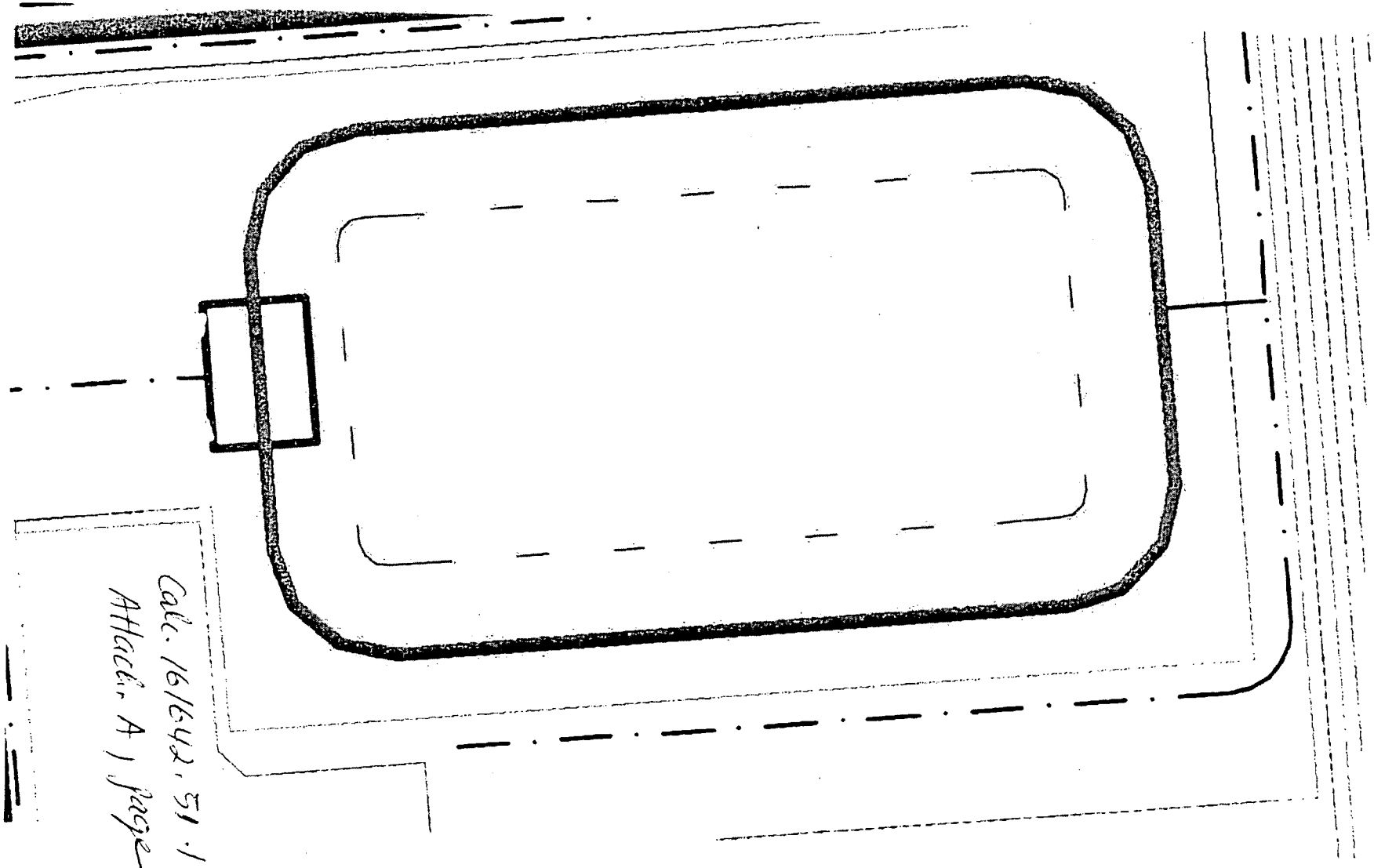
By DJW Date 10/1/08 Subject PMF for Bell Bend NPP Sheet No. 8 of 64
Chkd. by BXF Date 10/3/08 ESWEMS Pond Proj. No. 07-3891



Calc. 16/642.51.1512 R/C
Attach. A, page 9/9

ATTACHMENT B

Lag Time Calculation

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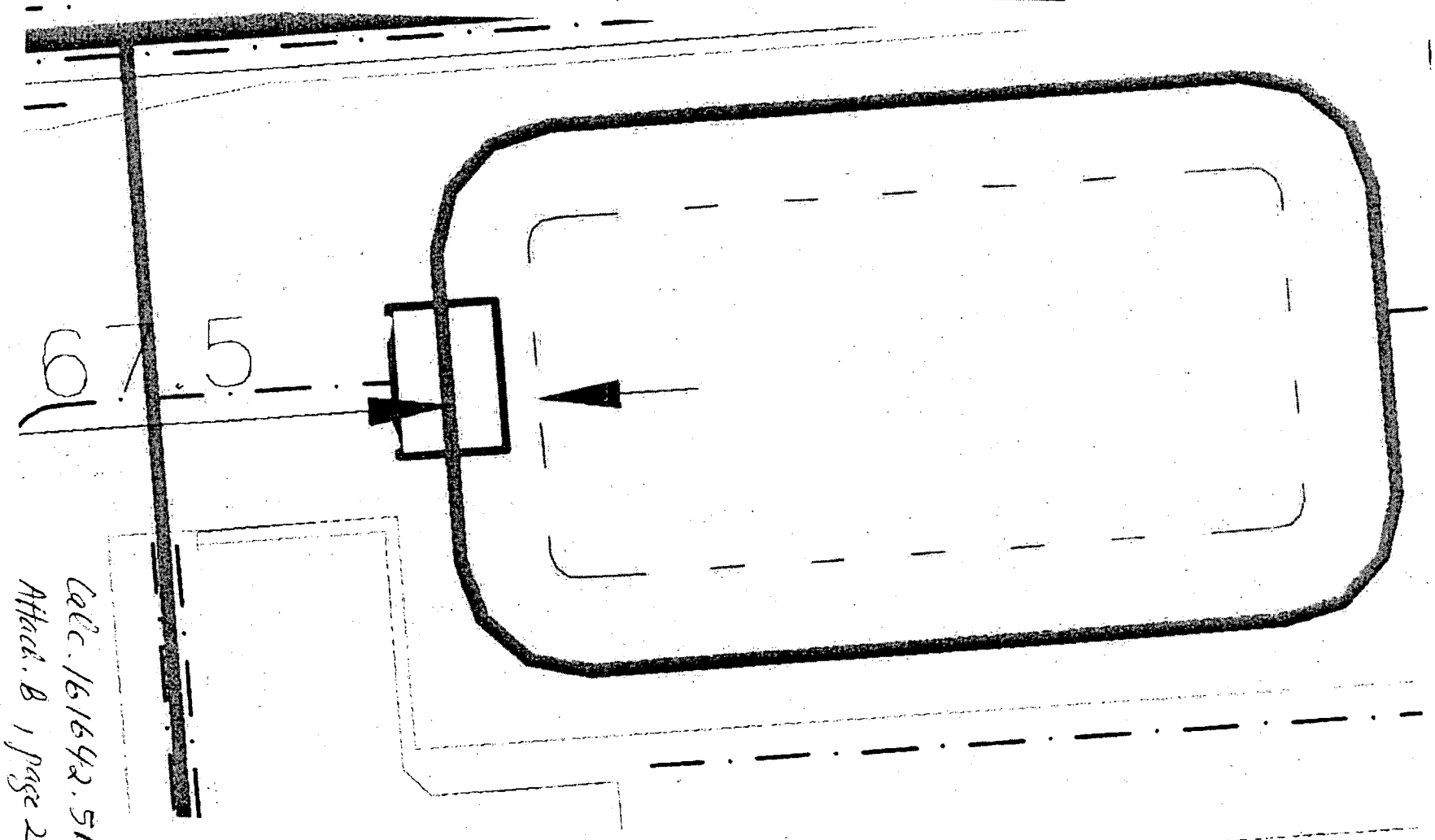
By DJW Date 10/1/08 Subject PMF for Bell Bend NPP Sheet No. 10 of 64
Chkd. by BKF Date 10/8/08 ESWEMS Pond Proj. No. 07-3891



Calc. 16164 2.51.1512/A/c
Attach. B 1 page 2/3

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39/d

By DJW Date 10/1/08 Subject PMF for Bell Bend NPP Sheet No. 11 of 64
Chkd. by BKF Date 10/8/08 ESWEMS Pond Proj. No. 07-3891

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project	Bell Bend NPP (07-3891)	By	BKW	Date	9/4/2008
Location	Berwick, PA	Checked		Date	

Check one: ☐ Present ☑ Developed (ESWEMS Pond)

Check one: ☐ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

$S = \frac{(674' - 651.5')}{67.5'} = 0.3333$

Sheet flow (Applicable to T_c only)

	Segment ID	1		
1. Surface description (table 3-1)		smooth, paved		
2. Manning's roughness coefficient, n (table 3-1)		0.011		
3. Flow length, L (total L ≯ 300 ft) ft		67.5		
4. Two-year 24-hour rainfall, P_2 in		3.0"		
5. Land slope, s ft/ft		0.3333		
6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\,s^{0.4}}$ Compute T_t hr		0.0049	+	= 0.0049

Shallow concentrated flow

	Segment ID			
7. Surface description (paved or unpaved)				
8. Flow length, L ft				
9. Watercourse slope, s ft/ft				
10. Average velocity, V (figure 3-1) ft/s				
11. $T_t = \frac{L}{3600\,V}$ Compute T_t hr			+	=

Channel flow

	Segment ID			
12. Cross sectional flow area, a ft^2				
13. Wetted perimeter, p_w ft				
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ft				
15. Channel slope, s ft/ft				
16. Manning's roughness coefficient, n				
17. $V = \frac{1.49\,r^{2/3}\,s^{1/2}}{n}$ Compute V ft/s				
18. Flow length, L ft				
19. $T_t = \frac{L}{3600\,V}$ Compute T_t hr			+	=
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) Hr				0.0049

= 0.297 min
× 0.6
$T_L \approx 1$ min

Attach B, page 3/3
Calc. 16/642.51.1512, R/0

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By DJW Date 10/1/08 Subject PMF for Bell Bend NPP Sheet No. 12 of 64
Chkd. by BKF Date 10/8/08 ESWEMS Pond Proj. No. 07-3891

ATTACHMENT C

Elevation-Storage Curve

Calc. 161642.51.1512 R/0
Attach. C, page 1/2

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By DJW . Date 10/1/08 . Subject PMF for Bell Bend NPP . Sheet No. 13 of 64 .
Chkd. by BLF . Date 10/3/08 . ESWEMS Pond . Proj. No. 07-3891 .

ESWEMS Pond Elevation-Storage Curve (Z = 3)

Elevation, El. (ft)	D (ft)	L (ft)	W (ft)	V (ft^3)	V (acre-ft)
651.5	0	565	265	0	**0**
652.5	1	565	265	152,227	**3.4945**
653.5	2	565	265	309,506	**7.1050**
654.5	3	565	265	471,909	**10.8332**
655.5	4	565	265	639,508	**14.6806**
656.5	5	565	265	812,375	**18.6489**
657.5	6	565	265	990,582	**22.7398**
658.5	7	565	265	1,174,201	**26.9550**
659.5	8	565	265	1,363,304	**31.2960**
660.5	9	565	265	1,557,963	**35.7646**
661.5	10	565	265	1,758,250	**40.3624**
662.5	11	565	265	1,964,237	**45.0911**
663.5	12	565	265	2,175,996	**49.9522**
664.5	13	565	265	2,393,599	**54.9475**
665.5	14	565	265	2,617,118	**60.0786**
666.5	15	565	265	2,846,625	**65.3472**
667.5	16	565	265	3,082,192	**70.7549**
668.5	17	565	265	3,323,891	**76.3033**
669.5	18	565	265	3,571,794	**81.9942**
670.5	19	565	265	3,825,973	**87.8291**
671.5	20	565	265	4,086,500	**93.8098**
672.5	21	565	265	4,353,447	**99.9378**
673.5	22	565	265	4,626,886	**106.2149**
674	22.5	565	265	4,766,063	**109.4099**

***BOLD columns are input to HEC-HMS for ESWEMS Pond Elevation-Storage Curve.**

Calc. 161642.51.1512
R/0, Attach C, page 2/2

G:\DJW\Berwick NPP (07-3891)\Model Based on Rev. 7 Drawings Dated 08-11-2008\Elevation-Storage Curves (08-11-08 Model)

ATTACHMENT D

HEC-HMS Output

Calc 16/642-51.1512
A/O, page 1/3

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By DJW Date 10/1/08 Subject PMF for Bell Bend NPP Sheet No. 15 of 64
Chkd. by BKF Date 10/3/08 ESWEMS Pond Proj. No. 07-3891

Summary Results for Subbasin "ESWEMS Pond Runoff"

Project : BBNPP Site Simulation Run : ESWEMS Pond ONLY Subbasin: ESWEMS Pond Runoff

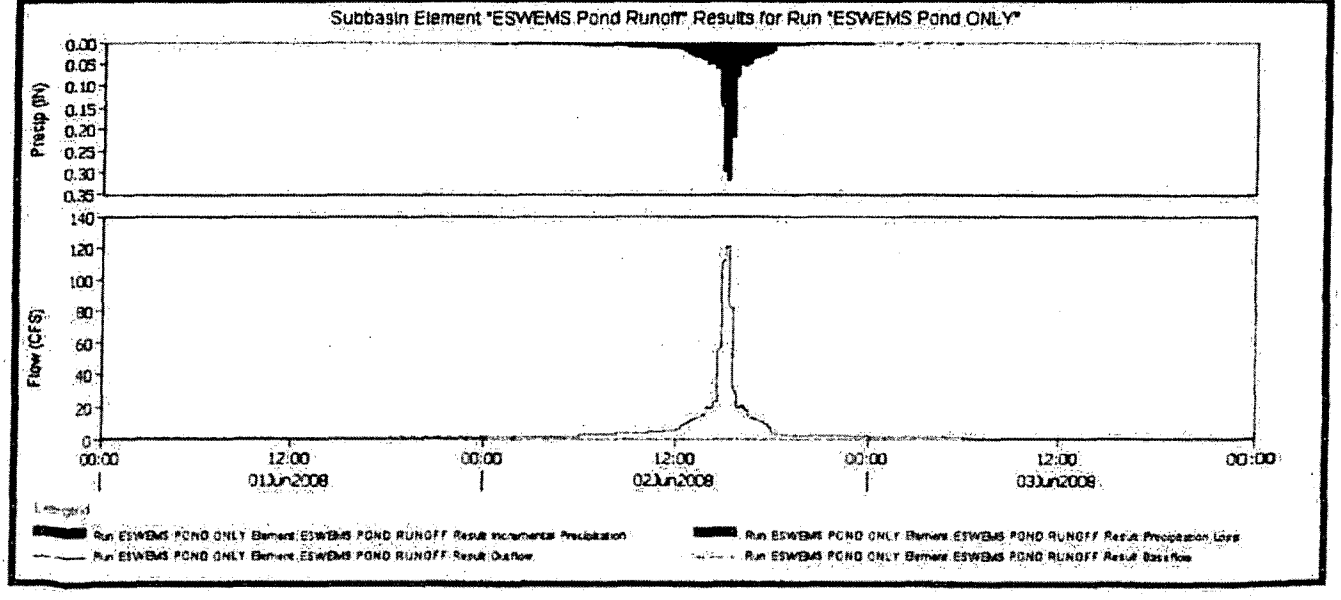
Start of Run :	01Jun2008, 00:00	Basin Model :	ESWEMS Pond ONLY
End of Run :	04Jun2008, 00:00	Meteorologic Model :	72-hr PMP
Compute Time :	11Sep2008, 09:42:50	Control Specifications :	72-hr PMF

Volume Units : (•) IN ( ) AC-FT

Computed Results

Peak Discharge :	120.71 (CFS)	Date/Time of Peak Discharge :	02Jun2008, 15:12
Total Precipitation :	37.77 (IN)	Total Direct Runoff :	37.77 (IN)
Total Loss :	0.00 (IN)	Total Baseflow :	0.00 (IN)
Total Excess :	37.77 (IN)	Discharge :	37.77 (IN)

Graph for Subbasin "ESWEMS Pond Runoff"



Calc. 1616 42.51.1512
R/0, Attach. D, page 2/3

Paul C. Rizzo Associates, Inc.
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By DJW Date 10/1/08 Subject PMF for Bell Bend NPP Sheet No. 16 of 64
Chkd. by BKF Date 10/8/08 ESWEMS Pond Proj. No. 07-3891

Summary Results for Reservoir "ESWEMS Pond Storage"

Project : BENPP Site Simulation Run : ESWEMS Pond ONLY Reservoir: ESWEMS Pond Storage

Start of Run :	01Jun2008, 00:00	Basin Model :	ESWEMS Pond ONLY
End of Run :	04Jun2008, 00:00	Meteorologic Model :	72-hr PMP
Compute Time :	11Sep2008, 09:42:50	Control Specifications :	72-hr PMF

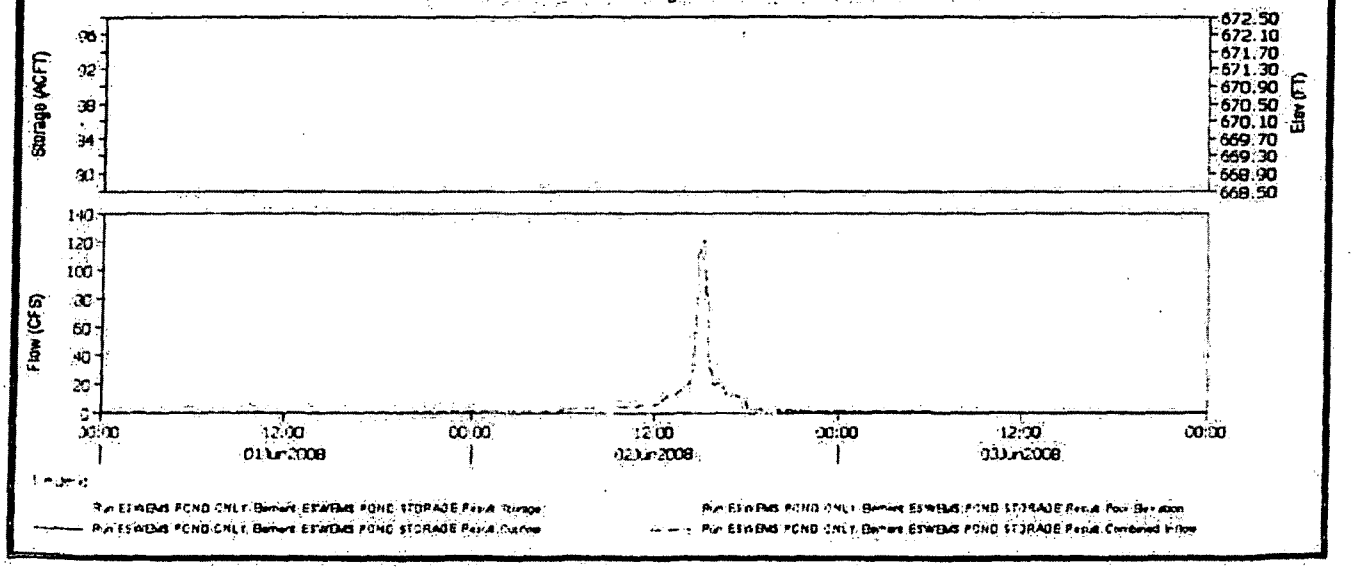
Volume Units : (•) IN. ( ) AC-FT

Computed Results

Peak Inflow :	120.71 (CFS)	Date/Time of Peak Inflow :	02Jun2008, 15:12
Peak Outflow :	0.71 (CFS)	Date/Time of Peak Outflow :	03Jun2008, 07:45
Total Inflow :	37.77 (IN)	Peak Storage :	97.6 (AC-FT)
Total Outflow :	2.50 (IN)	Peak Elevation :	672.1256 (FT)

Graph for Reservoir "ESWEMS Pond Storage"

Reservoir Element 'ESWEMS Pond Storage' Results for Run 'ESWEMS Pond ONLY'



Calc. 16/642.51.1512
R/0, Attach. D, page 3/3

Attachment 2
Black and Veatch Typical Riprap Specification

RIPRAP SPECIFICATION

Section 2C - RIPRAP

2C.1 GENERAL. This section covers materials and procedures for the installation of dumped riprap and riprap bedding on the ESWEMS pond slopes for wave protection. Riprap and riprap bedding shall be placed at the locations indicated on the drawings. Thickness of riprap and bedding shall be as indicated on the drawings.

Riprap and riprap bedding materials shall be obtained from a suitable offsite quarry. Alternatively, riprap may be obtained locally from the excavated rock if the specified gradations and material properties are met. The Contractor shall provide all hauling of these materials.

2C.2 CONTROL TESTING. Field placement control testing required to determine compliance with these specifications will be provided by the Owner. One copy of test results will be furnished to the Contractor.

2C.3 MATERIALS. Stone used for dumped riprap shall meet the gradation requirements specified and be hard, durable, angular in shape; resistant to weathering and to water action; and free from overburden, spoil, shale, and organic material. Rounded stones or boulders and shale and stones with shale seams shall not be used. The minimum unit weight of the stone shall be 162 pounds per cubic foot as computed by multiplying the specific gravity (bulk saturated surface dry basis, AASHTO T85) by 62.4 pounds per cubic foot.

The riprap bedding layers shall consist of gravel, crushed rock, sand, or a combination of these placed to the thickness indicated on the drawings. The gradation of material in the bedding layers shall meet the requirements specified. The riprap bedding layers shall be composed of tough, durable particles, free from thin, flat, and elongated pieces, and shall not contain organic matter or soft, friable particles in quantities in excess of those specified.

2C.3.1 Dumped Riprap. Each load of riprap shall be well graded from the smallest to the maximum size specified. Stones smaller than the specified nominal minimum size, spalls, sand, and rock dust shall not exceed 10 percent by weight of each load.
The dumped riprap gradation requirements shall be as follows unless otherwise acceptable to the Engineer:

Nominal Stone Size in pounds	Percent of Total Weight, Smaller than the Given Size
500 (18 inch)	100
350	92 - 98
220	75 - 85
116 (12 inch)	45 - 55(D_{50})
15	< 10 (Nominal Minimum)

If a question of material suitability arises, the following test shall be performed by the Owner to determine material acceptability:

Test	Designation	Requirements
Specific gravity (bulk saturated surface dry)	AASHTO T85	Greater than or equal to 2.60.
Abrasion (Abrasive Grading A)	AASHTO T96	Less than 40 percent loss of weight after 500 revolutions.
Freezing and thawing (ledge rock type test and sample tested by Procedure A)	AASHTO T103	Less than 10 percent loss after 12 cycles.

2C.3.2 Riprap Bedding. Riprap bedding and gradation requirements shall be as follows unless otherwise acceptable to the Engineer:

Coarse Riprap Bedding	
U.S. Standard Sieve Size	Percent Passing
2 inch	100
1 1/2 inch	95 - 100
3/4 inch	35 - 70

3/8 inch	10-30
No. 4	0 - 5
Note: Department of Transportation standard aggregates with this approximate gradation may be submitted for consideration.	

Fine Riprap Bedding	
U.S. Standard Sieve Size	Percent Passing
3/8 inch	100
# 4	95 - 100
# 8	80 - 100
# 16	50 - 85
# 30	25 - 60
# 50	10 - 30
# 100	0 - 10
Note: Department of Transportation standard aggregates with this approximate gradation may be submitted for consideration.	

2C.4 REFERENCE SAMPLES. Gradation control of dumped riprap will be by visual inspection. The Construction Manager will accompany the Contractor to the quarry to inspect a 5 ton sample of stone meeting the gradation. The sample shall be used as a reference for judging the gradation of the riprap placed. This inspection shall be completed before riprap work begins. Disputes over gradation shall be resolved by dumping and checking the gradation of two random truckloads of stone. The Construction Manager will check the gradation of this stone. Equipment and labor needed to assist the checking of gradation shall be provided.

2C.5 PRELIMINARY REVIEW. The Owner's acceptance of the source and quality of riprap material shall be obtained before beginning the riprap work. Certified reports shall be submitted to the Engineer in accordance with (later). The reports shall certify compliance with the requirements of these specifications. Continued compliance with all contract provisions will be required.

The stone sources shall be selected well in advance of the scheduled time for riprap construction work. Stone acceptability shall be determined by testing. Suitable samples of stone shall be taken in the presence of the Owner at least 25 days before riprap work is

scheduled to begin. The acceptance of some rock fragments from a quarry site shall not construe acceptance of all rock fragments from that quarry.

2C.5.2 Riprap Bedding. Riprap bedding materials shall meet the following requirements:

Test	Designation	Requirements
Sampling	AASHTO T2	No special requirements.
Sieve analysis	AASHTO T27	Percentages passing standard sieve sizes provided in the coarse and fine bedding tables.
Organic matter (Alternate Procedure A)	AASHTO T21	Test solution lighter in color than standard.
Clay lumps	AASHTO T112	Not to exceed 1.5 percent by weight.
Lightweight pieces	AASHTO T113	Not to exceed 2 percent by weight.

If available, documented service records of the proposed material shall be furnished to the Owner for evaluating acceptability of the stone.

The bedding materials shall meet the specific gravity, abrasion, and freeze and thaw test requirements specified as the dumped riprap.

2C.6 PLACEMENT. Riprap work shall not start until the riprap materials are accepted by the Owner. Dumped riprap and riprap bedding materials shall be placed where indicated on the drawings.

Earth slopes shall be prepared and compacted as specified in the section titled EARTHWORK. Riprap bedding layers shall be placed on the prepared areas to the full layer thickness in one operation without causing segregation of particle sizes. Additional layers shall be placed without mixing the material between layers. The finished surface shall be even and free from mounds or windrows.

Riprap shall be placed on the prepared bedding areas to produce a well graded mass of stone with a minimum percentage of voids. The entire stone mass shall be placed to the lines and grades indicated on the drawings. Riprap shall be placed to the indicated

thickness in one operation without displacing the underlying material. Riprap shall be placed without segregation of material.

Large stones shall be well distributed and the entire stone mass shall conform to the specified gradation. Riprap protection shall be placed and distributed to avoid large accumulations of either the larger or smaller sizes of stone.

The finished riprap shall be uniformly distributed so that the smaller rock fragments fill the spaces between the larger rock fragments resulting in a compact, uniform layer of the specified thickness.

The lines and grades indicated on the drawings shall be provided within a tolerance of 6 inches. Deviations from the designated lines, at the limits of the specified tolerance, shall not extend over an area greater than 200 square feet.