ATTACHMENT 23
Surface Waters Control
Special Waste Landfill Permit
Big Sandy Plant – Ash Pond Closure
Lawrence County, Kentucky

During Construction

An application for coverage under the KPDES permit program for storm water associated with construction activities will be submitted to the KDEP prior to construction activities. A storm water pollution prevention plan will be developed and best management practices will be implemented outlined in *Kentucky Best Management Practices for Construction Activities* during the construction phases of the closure project to avoid or minimize impacts to storm water. For example, straw bales and rock check dams will be provided at selected locations for sediment trapping and deposition. Vegetation will be established in accordance with the closure cap specifications in order to stabilize the final cover soils and seedbed preparation, and when seasonal conditions are suitable for the type of vegetation to be used. In general, as much of the exposed soil as practical will be seeded prior to September of each year in order to establish a vegetative cover prior to significant rainfall events that typically occur in the Spring of the following year.

Post Construction

The proposed closure cap design is to include engineered tributaries and primary channel at a maximum 5 percent inverted grade. The capped area will be graded at 2% (typical) to the channels. The slopes and channels will be appropriately vegetated and/or lined to mitigate erosion.

No other post-construction storm water controls will be required for this project. As shown by the associated calculations provided, the pond closure design would be able to support flows through the affected basin for the 100 year, 24 hour event. Associated calculations have been provided as a part of this attachment.

OBJECTIVE:

Develop the frequency event hydrology for Big Sandy.

REFERENCES:

-Department for Natural Resources and Environmental Protection, Division of Water, *Design Criteria for Dams and Associated Structures (Engineering Memorandum Number 5)*, 401 KAR 4:030, June 1999.

-ESRI, *ArcGIS Desktop 10 Service Pack 5*, 2010.

-Kentucky Power Company (KPC), *Big Sandy Fly Ash Dam Stage 3 Raising Engineering Report*, 1990.

-National Resources Conservation Service (NRCS), *National Engineering Handbook*, May 2010.

-NOAA Atlas 14, Precipitation Frequency Data Server, Volume 2, Version 3, Data Extracted July 2012.

-U.S. Army Corps of Engineers (USACE), *Hydrologic Modeling System (HEC-HMS)*, Version: 3.5, Build: 1417, August 10, 2010.

-U.S. Department of Agriculture (USDA), Soil Conservation Service, *SCS National Engineering Handbook*, Section 4: Hydrology, 1964, updated 1972.

FILE LOCATION:

The contents of this calculation are located at the following network address:

T:\Projects\Glenwood\AEP Big Sandy\CALCULATIONS\Frequency Event Hydrology

APPROACH:

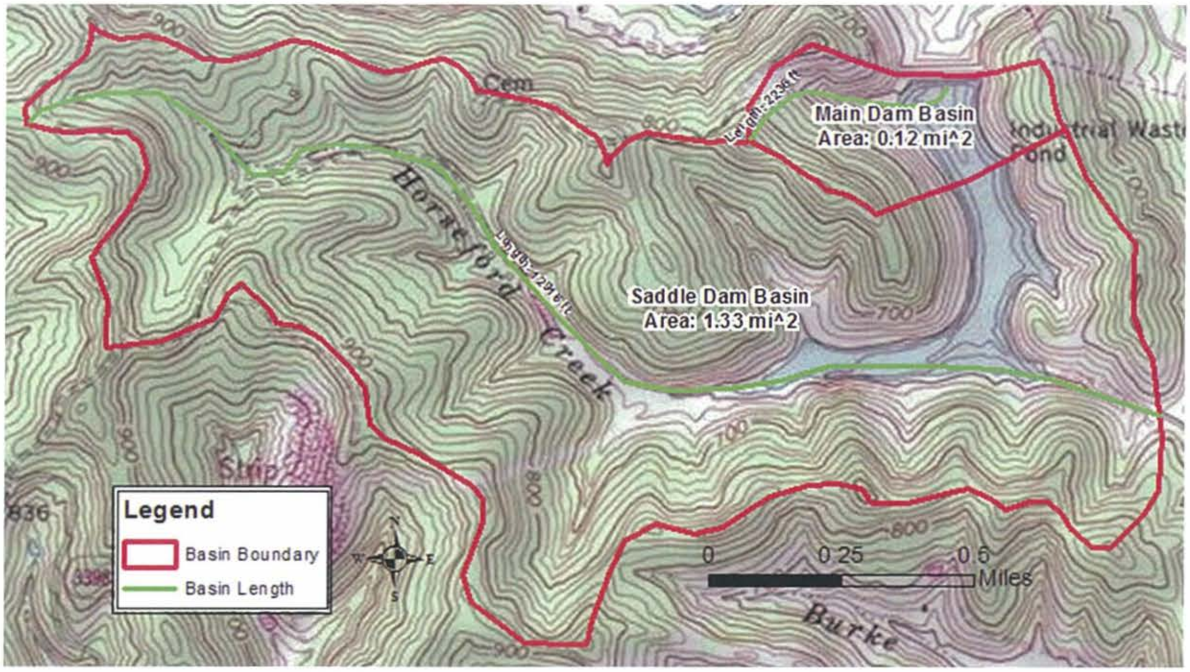
To develop the frequency hydrology for the basins, the following steps were taken:

Step 1: Delineate the basins
Step 2: Estimate hydrologic characteristics
Step 3: Calculate runoff characteristics

These steps are discussed in detail below.

<u>Step 1: Delineate the basins</u>

Using GIS (ESRI 2010) the basins were delineated. USGS topography was downloaded which features 20 foot contours. Two (2) basins resulted, namely the 1) Main Dam and the 2) Saddle Dam. The figure show below shows these basins.



Delineated Basins (includes calculated attributes)

Step 2: Estimate hydrologic characteristics of basins

To develop the hydrology of the basins using the well accepted National Resource Conservation Service (NRCS) methodology, the following characteristics are required: the a) basin area, b) slope, c) CN number, d) lag time, e) time of concentration, f) initial abstraction, and g) precipitation. See attached for the calculation spreadsheet associated with the discussion below.

a) Basin area:
The basin areas were estimated using GIS (ESRI 2010). Note the estimated areas shown in the above figure. The area of the Main Dam and Saddle Dam basins are 0.12 and 1.33 square miles respectfully.

b) Basin slope:
The basin average land slopes were estimated using the equation (eq. 15-6) presented in the NRCS methodology (NRCS 2010). Required input for this equation included the summation of the length of contour lines that pass through the basin, the contour interval used, and the basin area. These parameters were estimated using GIS (see file for more detail). The average land slopes for the Main Dam and Saddle Dam basins were estimated to be 34.9 and 24.9 percent respectfully.

c) CN number:
The weighted CN number was calculated for the two basins. In accordance with Kentucky regulation (Department for Natural Resources and Environmental Protection 1999), antecedent moisture conditions II (AMC II) was used. For the Main Dam basin, following Figure 9-1 (NRCS

2010), the basin cover consists of approximate 50 percent of Oak-Aspen. Soil group C was assumed representative of the basin which is consistent with a previous report (KPC 1990). The ground cover density is approximately 80 percent resulting in CN numbers of 44 (see Figure 9-1, NRCS 2010). Assuming the remaining 50 percent of the area consists of 25 percent bare soil type C (Table 9-1, CN = 91) and 25 percent water due to possible ponding (CN = 100), the weighted CN value for the basin is 69.8.
For the Saddle Dam basin, following Figure 9-1 (NRCS 2010), the basin cover consists of approximate 50 percent of Oak-Aspen. Soil group C was assumed representative of the basin which is consistent with a previous report (KPC 1990). The ground cover density is approximately 85 percent resulting in CN numbers of 40 (see Figure 9-1, NRCS 2010). Assuming the remaining 50 percent of the area consists of 50 percent bare soil type C (Table 9-1, CN = 91), the weighted CN value for the basin is 65.5.

d) Lag time:
The lag time was estimated by using the equation eq. 15-4a found in the NRCS methodology (NRCS 2010). Input for this equation included the flow length and average watershed land slope. The flow lengths were approximated using GIS (ESRI 2010) to be 2,236 and 12,916 feet for the Main Dam and Saddle Dam basins respectfully (see figure above). The average watershed land slopes were approximated previously (see step b). The lag time for the Main Dam and Saddle Dam Basins was estimated to be 8 and 44 minutes respectively.

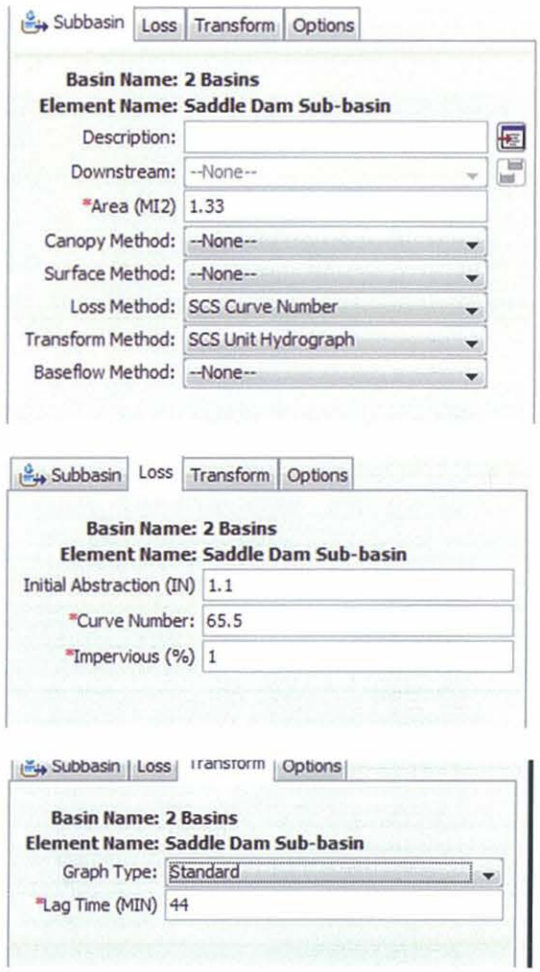
e) Time of concentration:
The time of concentration was estimated using equation eq. 15-3 found in the NRCS methodology (NRCS 2010). There is only one variable in this equation which is the lag time calculated in the previous step (step d). The time of concentration for the Main Dam and Saddle Dam basin is approximately 14 and 74 minutes respectfully.

f) Initial abstraction:
In accordance with the USDA (1972) and NRCS (2010) guidelines the initial abstraction was estimated. The maximum potential retention (S) was determined using the equation eq. 15-4b (see variable definitions to equation) (NRCS 2010). The initial abstraction (Ia) was calculated as Ia = 0.2S (USDA 1972) yielding 0.9 and 1.1 inches for the Main Dam and Saddle Dam basins respectfully.

g) Precipitation:
Precipitation values were taken from the NOAA Atlas 14 database (NOAA 2012). See attached reference material for the specific values used.

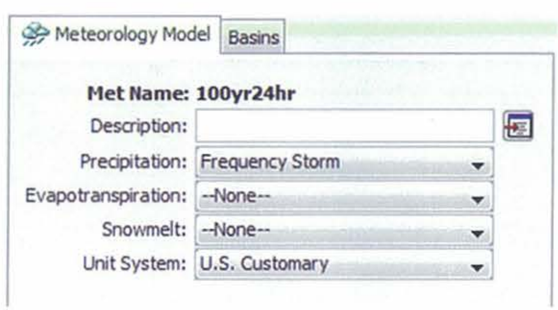
<u>Step 3: Calculate runoff characteristics</u>

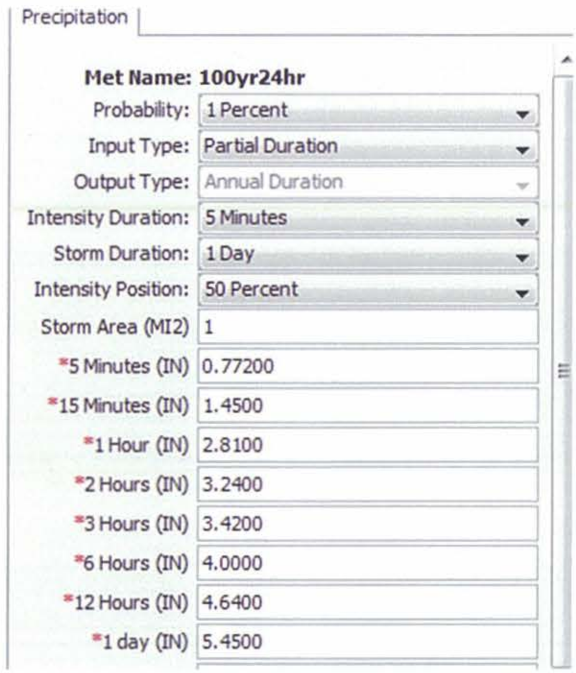
A HEC-HMS (USACE 2010) model was built to estimate the runoff characteristics of each basin. The runoff characteristics were estimated by inputting the above computed parameters into HEC-HMS. As an example of the setup, screenshots of the Saddle Dam input are shown below.



Basin Setup Example (Saddle Dam)

The meteorologic models were setup up for the 2, 10, 25, 100 year frequency events for both 6 and 24 hour durations. As an example, the 100 year 24 hour duration input is shown in the screenshot below.

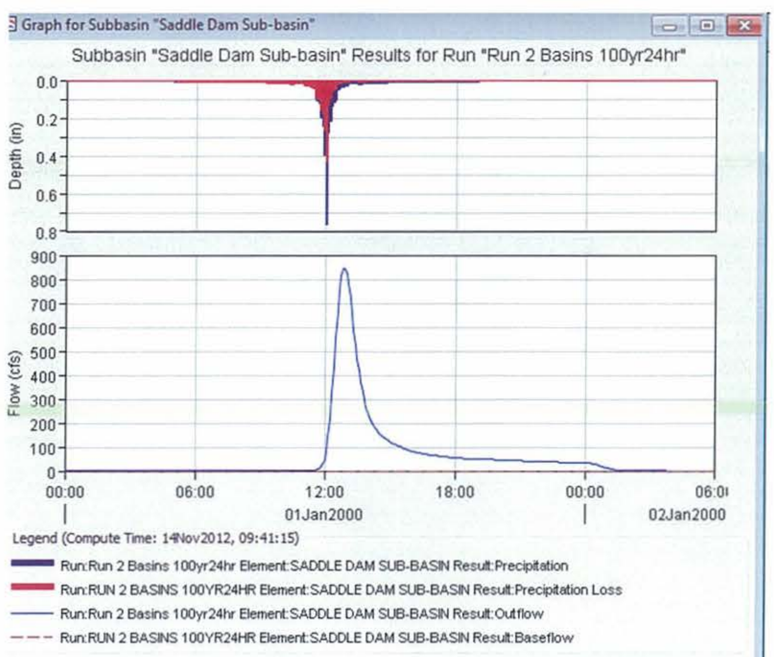




Meteorologic Model Setup Example (100 year frequency 24 hour duration event)

RESULTS:

As a graphical example of the results is shown below for the Saddle Dam basin 100 year frequency 24 hour duration event.



Graphical Example of Results (Saddle Dam basin 100 year frequency 24 hour duration event)

The peak discharge for each of the studied events is presented in the table below.

Results Table

Frequency Event	Duration	Peak Discharge: Saddle Dam Basin	Peak Discharge: Main Dam Basin
yr	hr	cfs	cfs
100	6	570	153
100	24	844	216
25	6	321	96
25	24	523	146
10	6	188	61
10	24	337	101
2	6	25	10
2	24	72	29

ATTACHED:

-Precipitation Data (NOAA 2012)

-Calculation reference material (NRCS 2010, KPC 1990, Department for Natural Resources and Environmental Protection 1999)

Big Sandy

Drainage Basin	Area (ft^2)	Area (acres)	Area (mi^2)	Length (ft)	Slope (%)	CN	S (in)	Initial Abstraction (in)	Lag Time (hr)	Lag Time (min)	Time of Concentration (hr)	Time of Concentration (min)
Saddle Dam	37077957	851.2	1.33	12916	24.9	65.5	5.3	1.1	0.7	44	1.2	74
Main Dam	3345459	76.8	0.12	2236	34.9	69.8	4.3	0.9	0.1	8	0.2	14

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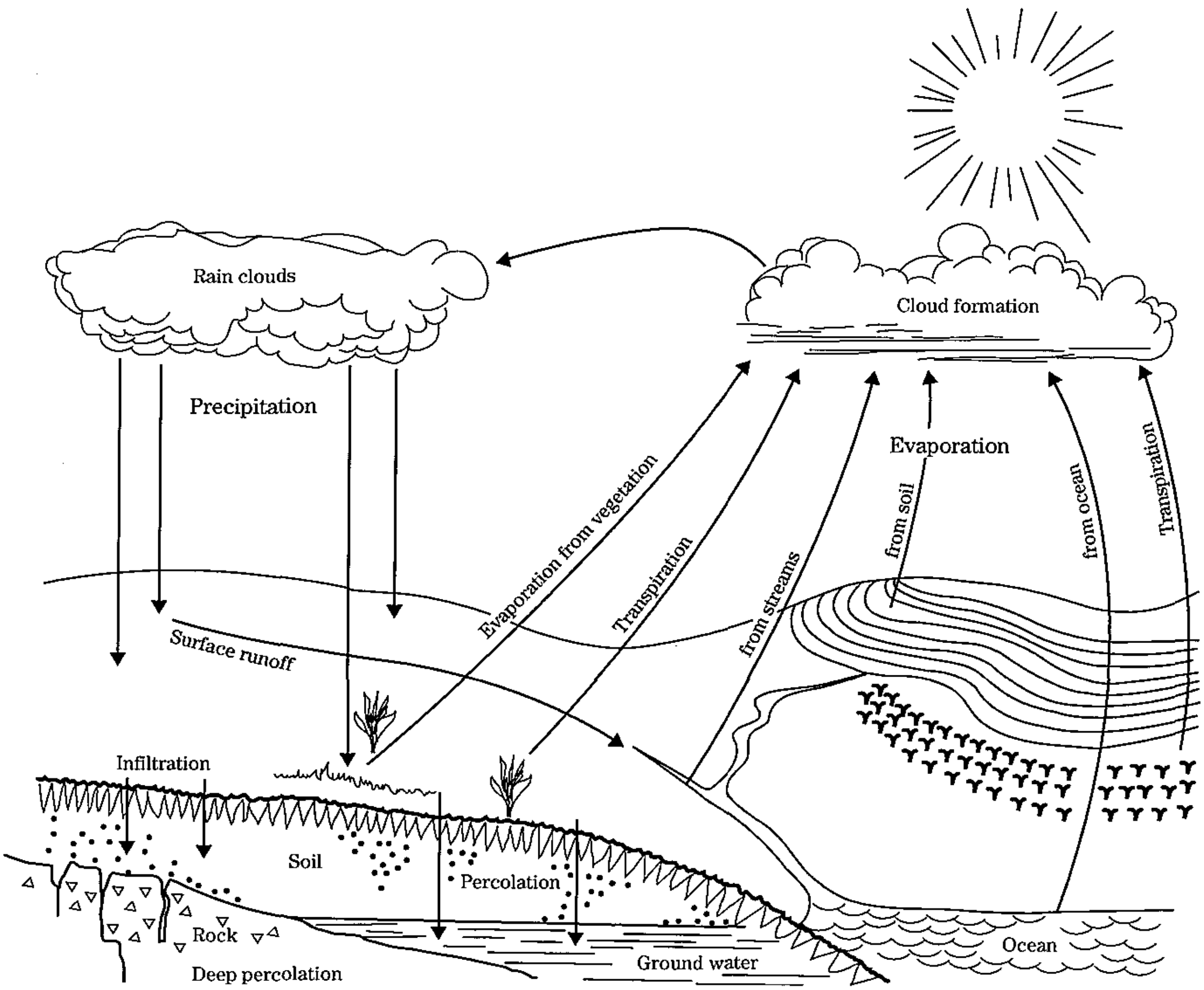
Chapter 9 Hydrologic Soil-Cover Complexes



(2) Use of table 9–1

Chapters 7 and 8 of NEH 630 describe how soils and covers of watersheds or other land areas are classified in the field. After the classification is completed, CNs are read from table 9–1 and applied as described in chapter 10. Because the principal use of CNs is for estimating runoff from rainfall, the examples of applications are given in chapter 10.

Table 9–1 Runoff curve numbers for agricultural lands [1/]

Cover description			CN for hydrologic soil group			
cover type	treatment [2/]	hydrologic condition [3/]	A	B	C	D
Fallow	Bare Soil	---	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C & T)	Poor	66	74	80	82
		Good	62	71	78	81
	C & T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C & T	Poor	61	72	79	82
		Good	59	70	78	81
	C & T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C & T	Poor	63	73	80	83
		Good	51	67	76	80

See footnotes at end of table.

Table 9–1 Runoff curve numbers for agricultural lands [1] — Continued

Cover description: cover type	treatment [2]	hydrologic condition [3]	CN for hydrologic soil group: A	B	C	D
Pasture, grassland, or range-continuous forage for grazing [4]		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
Meadow-continuous grass, protected from grazing and generally mowed for hay		Good	30	58	71	78
Brush-brush-forbs-grass mixture with brush the major element [5]		Poor	48	67	77	83
		Fair	35	56	70	77
		Good	30 [6]	48	65	73
Woods-grass combination (orchard or tree farm) [7]		Poor	57	73	82	86
		Fair	43	65	76	82
		Good	32	58	72	79
Woods [8]		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	30	55	70	77
Farmstead--buildings, lanes, driveways, and surrounding lots		---	59	74	82	86
Roads (including right-of-way):						
Dirt		---	72	82	87	89
Gravel		---	76	85	89	91

1/ Average runoff condition, and I_a=0.2s.

2/ Crop residue cover applies only if residue is on at least 5 percent of the surface throughout the year.

3/ Hydrologic condition is based on combinations of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥20%), and (e) degree of surface toughness.
Poor: Factors impair infiltration and tend to increase runoff.
Good: Factors encourage average and better then average infiltration and tend to decrease runoff.
For conservation tillage poor hydrologic condition, 5 to 20 percent of the surface is covered with residue (less than 750 pounds per acre for row crops or 300 pounds per acre for small grain).
For conservation tillage good hydrologic condition, more than 20 percent of the surface is covered with residue (greater than 750 pounds per acre for row crops or 300 pounds per acre for small grain).

4/ Poor: < 50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

5/ Poor: < 50% ground cover.
Fair: 50 to 75% ground cover.
Good: > 75% ground cover.

6/ If actual curve number is less than 30, use CN = 30 for runoff computation.

7/ CNs shown were computed for areas with 50 percent woods and 50 percent grass (pasture) cover. Other combinations of conditions may be computed from the CNs for woods and pasture.

8/ Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed, but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

(b) National and commercial forest: forest-range

(1) Forest-range in Western United States

In the forest-range regions of the Western United States, soil group, cover type, and cover density are the principal factors used in estimating CNs. Figures 9–1 and 9–2 show the relationships between these factors and CNs for soil-cover complexes used to date. The figures are based on information in table 2–1, part 2, of the USDA Forest Service's Handbook on Methods of Hydrologic Analysis (USDA 1959b). The amount of litter is taken into account when estimating the density of cover.

Present hydrologic conditions are determined from existing surveys or by reconnaissance, and future conditions from the estimate of cover and density changes resulting from proper use and treatment. Table 9–2 lists CNs for arid and semiarid rangelands. It is used like table 9–1.

Figure 9–1 Estimating runoff curve numbers of forest-range complexes in Western United States: herbaceous and oak-aspen complexes

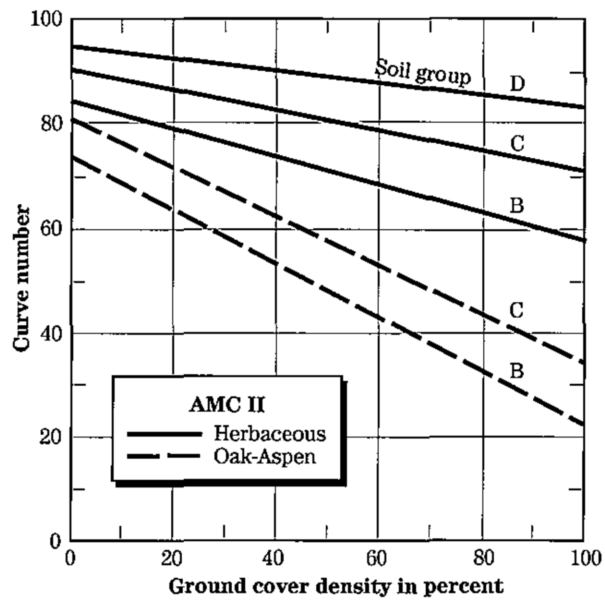
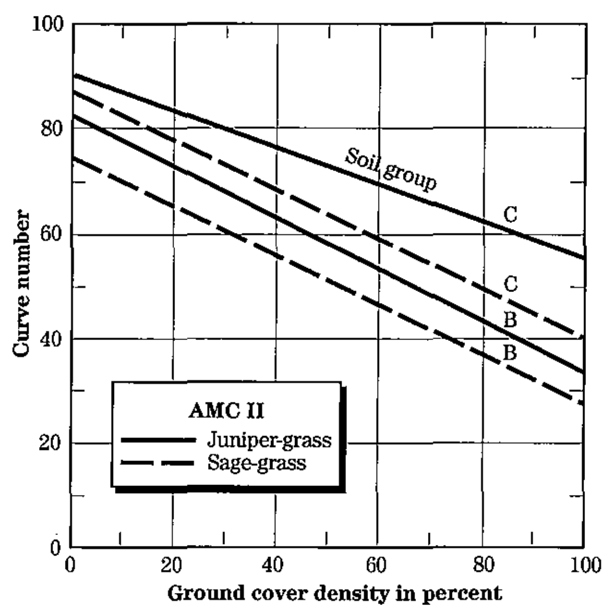


Figure 9–2 Estimating runoff curve numbers of forest-range complexes in Western United States: juniper-grass and sage-grass complexes



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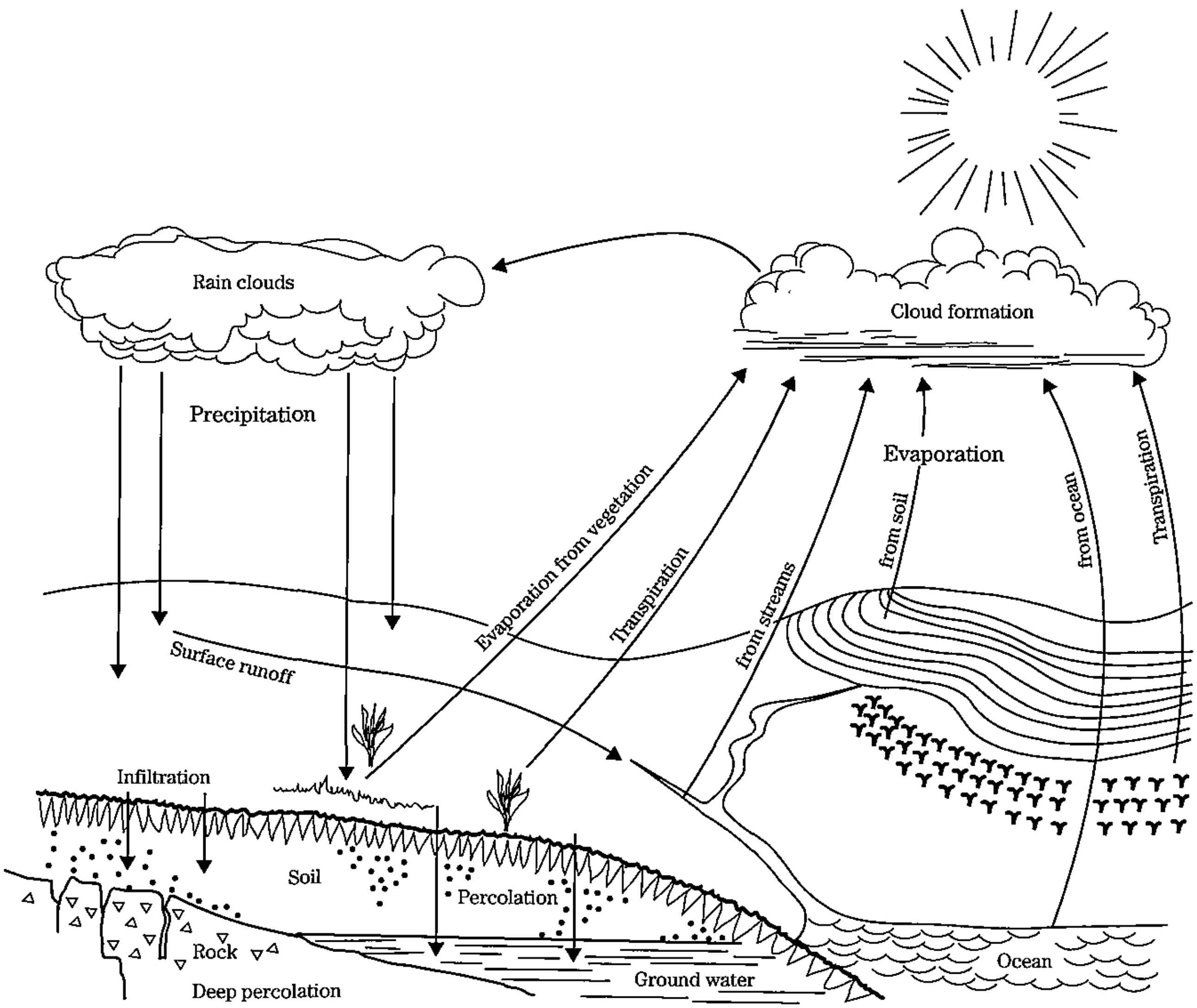
Chapter 15 Time of Concentration



of concentration for that single area is required. A hydrograph is then developed using the methods described in NEH630.16. However, if land use, hydrologic soil group, slope, or other watershed characteristics are not homogeneous throughout the watershed, the approach is to divide the watershed into a number of smaller subareas, which requires a time of concentration estimation for each subarea. Hydrographs are then developed for each subarea by the methods described in NEH630.16 and routed appropriately to a point of reference using the methods described in NEH630.17, Flood Routing.

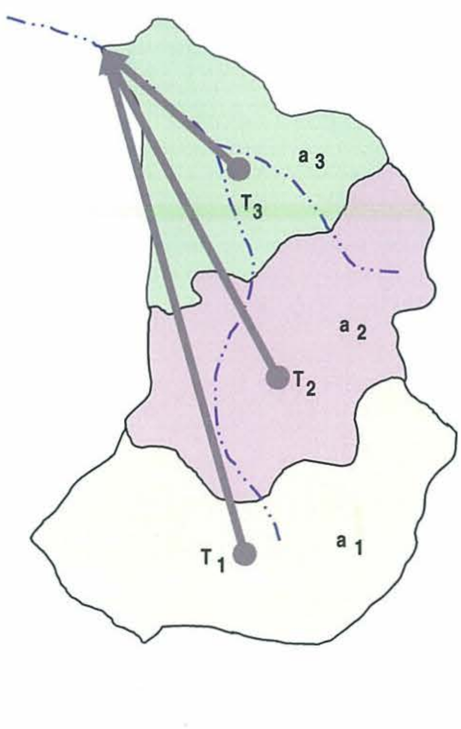
In hydrograph analysis, lag is the time interval between the center of mass of the excess rainfall and the peak runoff rate (fig. 15–3).

(d) Time of concentration

Time of concentration (T_c) is the time required for runoff to travel from the hydraulically most distant point in the watershed to the outlet. The hydraulically most distant point is the point with the longest travel time to the watershed outlet, and not necessarily the point with the longest flow distance to the outlet. Time of concentration is generally applied only to surface runoff and may be computed using many different methods. Time of concentration will vary depending upon slope and character of the watershed and the flow path.

In hydrograph analysis, time of concentration is the time from the end of excess rainfall to the point on the falling limb of the dimensionless unit hydrograph (point of inflection) where the recession curve begins (fig. 15–3).

Figure 15–2 Conceptual watershed illustrating travel time from the centroid (gray dot) of each band of area to the watershed outlet



(e) Relation between lag and time of concentration

Various researchers (Mockus 1957; Simas 1996) found that for average natural watershed conditions and an approximately uniform distribution of runoff:

$$L = 0.6T_c \qquad \text{(eq. 15–3)}$$

where:
L = lag, h
T_c = time of concentration, h

When runoff is not uniformly distributed, the watershed can be subdivided into areas with nearly uniform flow so that equation 15–3 can be applied to each of the subareas.

630.1502 Methods for estimating time of concentration

Two primary methods of computing time of concentration were developed by the Natural Resources Conservation Service (NRCS) (formerly the Soil Conservation Service (SCS)).

(a) Watershed lag method

The SCS method for watershed lag was developed by Mockus in 1961. It spans a broad set of conditions ranging from heavily forested watersheds with steep channels and a high percent of runoff resulting from subsurface flow, to meadows providing a high retardance to surface runoff, to smooth land surfaces and large paved areas.

$$L = \frac{\ell^{0.8}(S+1)^{0.7}}{1{,}900Y^{0.5}} \qquad \text{(eq. 15–4a)}$$

Applying equation 15–3, $L=0.6T_c$, yields:

$$T_c = \frac{\ell^{0.8}(S+1)^{0.7}}{1{,}140Y^{0.5}} \qquad \text{(eq. 15–4b)}$$

where:
L = lag, h
T_c = time of concentration, h
ℓ = flow length, ft
Y = average watershed land slope, %
S = maximum potential retention, in

$$= \frac{1{,}000}{cn'} - 10$$

where:
cn' = the retardance factor

Flow length (ℓ)—In the watershed lag method of computing time of concentration, flow length is defined as the longest path along which water flows from the watershed divide to the outlet. In developing the regression equation for the lag method, the longest flow path was used to represent the hydraulically most distant point in the watershed. Flow length can be measured using aerial photographs, quadrangle sheets, or GIS techniques. Mockus (USDA 1973) developed an empirical relationship between flow length and drainage area using data from Agricultural Research Service (ARS) watersheds. This relationship is:

$$\ell = 209A^{0.6} \qquad \text{(eq. 15–5)}$$

where:
ℓ = flow length, ft
A = drainage area, acres

Land slope (Y), percent—The average land slope of the watershed, as used in the lag method, not to be confused with the slope of the flow path, can be determined in several different ways:

- by assuming land slope is equal to a weighted average of soil map unit slopes, determined using the local soil survey
- by using a clinometer for field measurement to determine an estimated representative average land slope
- by drawing three to four lines on a topographic map perpendicular to the contour lines and determining the average weighted slope of these lines
- by determining the average of the land slope from grid points using a dot counter
- by using the following equation (Chow 1964):

$$Y = \frac{100(CI)}{A} \qquad \text{(eq. 15–6)}$$

where:
Y = average land slope, %
C = summation of the length of the contour lines that pass through the watershed drainage area on the quad sheet, ft
I = contour interval used, ft
A = drainage area, ft^2 (1 acre = 43,560 ft^2)

Retardance factor—The retardance factor, cn′, is a measure of surface conditions relating to the rate at which runoff concentrates at some point of interest. The term "retardance factor" expresses an inverse relationship to "flow retardance." Low retardance factors are associated with rough surfaces having high degrees of flow retardance, or surfaces over which flow will be impeded. High retardance factors are associated with smooth surfaces having low degrees of flow retardance, or surfaces over which flow moves rapidly.

NOAA Atlas 14, Volume 2, Version 3
Location name: Louisa, Kentucky, US*
Coordinates: 38.1800, -82.6350
Elevation: 672ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval(years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.357 (0.327–0.390)	0.425 (0.389–0.467)	0.508 (0.465–0.557)	0.572 (0.522–0.625)	0.653 (0.593–0.711)	0.714 (0.646–0.778)	0.772 (0.696–0.840)	0.831 (0.747–0.902)	0.906 (0.809–0.983)	0.961 (0.853–1.04)
10-min	0.556 (0.509–0.607)	0.665 (0.609–0.731)	0.791 (0.725–0.867)	0.885 (0.807–0.968)	1.00 (0.909–1.09)	1.09 (0.982–1.18)	1.17 (1.05–1.27)	1.24 (1.12–1.35)	1.34 (1.19–1.45)	1.40 (1.25–1.52)
15-min	0.682 (0.624–0.745)	0.814 (0.745–0.894)	0.972 (0.890–1.07)	1.09 (0.994–1.19)	1.24 (1.12–1.35)	1.34 (1.22–1.47)	1.45 (1.31–1.58)	1.55 (1.39–1.68)	1.67 (1.49–1.81)	1.75 (1.56–1.90)
30-min	0.905 (0.828–0.987)	1.09 (1.00–1.20)	1.33 (1.22–1.46)	1.52 (1.38–1.66)	1.75 (1.59–1.91)	1.93 (1.74–2.10)	2.10 (1.89–2.28)	2.27 (2.04–2.46)	2.48 (2.21–2.69)	2.64 (2.34–2.87)
60-min	1.11 (1.01–1.21)	1.34 (1.23–1.47)	1.68 (1.54–1.84)	1.93 (1.76–2.11)	2.27 (2.07–2.48)	2.54 (2.30–2.77)	2.81 (2.53–3.05)	3.08 (2.77–3.34)	3.43 (3.07–3.72)	3.71 (3.30–4.03)
2-hr	1.26 (1.15–1.37)	1.52 (1.40–1.66)	1.90 (1.74–2.08)	2.19 (2.00–2.39)	2.60 (2.36–2.83)	2.91 (2.64–3.16)	3.24 (2.92–3.51)	3.57 (3.20–3.86)	4.02 (3.58–4.34)	4.37 (3.87–4.73)
3-hr	1.32 (1.21–1.45)	1.60 (1.46–1.76)	1.99 (1.83–2.19)	2.30 (2.10–2.52)	2.73 (2.47–2.98)	3.07 (2.77–3.35)	3.42 (3.08–3.72)	3.78 (3.39–4.10)	4.28 (3.80–4.63)	4.67 (4.12–5.05)
6-hr	1.57 (1.45–1.71)	1.88 (1.73–2.05)	2.32 (2.14–2.53)	2.67 (2.45–2.91)	3.17 (2.90–3.45)	3.58 (3.25–3.87)	4.00 (3.61–4.31)	4.43 (3.98–4.77)	5.04 (4.47–5.40)	5.52 (4.87–5.90)
12-hr	1.86 (1.73–2.01)	2.21 (2.06–2.40)	2.70 (2.51–2.92)	3.11 (2.88–3.35)	3.68 (3.40–3.96)	4.15 (3.81–4.45)	4.64 (4.23–4.96)	5.15 (4.67–5.50)	5.87 (5.27–6.25)	6.46 (5.73–6.86)
24-hr	2.27 (2.12–2.43)	2.71 (2.52–2.90)	3.27 (3.05–3.51)	3.73 (3.48–4.00)	4.38 (4.07–4.69)	4.91 (4.54–5.24)	5.45 (5.04–5.82)	6.02 (5.54–6.42)	6.81 (6.23–7.24)	7.44 (6.77–7.90)
2-day	2.73 (2.55–2.92)	3.23 (3.03–3.46)	3.88 (3.63–4.14)	4.39 (4.10–4.69)	5.10 (4.75–5.44)	5.66 (5.27–6.04)	6.24 (5.79–6.65)	6.84 (6.32–7.28)	7.65 (7.04–8.13)	8.29 (7.60–8.81)
3-day	2.93 (2.74–3.13)	3.47 (3.25–3.71)	4.13 (3.87–4.42)	4.67 (4.36–4.98)	5.38 (5.02–5.74)	5.95 (5.54–6.35)	6.53 (6.07–6.95)	7.12 (6.59–7.57)	7.91 (7.29–8.40)	8.52 (7.83–9.05)
4-day	3.13 (2.93–3.34)	3.70 (3.47–3.95)	4.39 (4.11–4.69)	4.94 (4.62–5.28)	5.67 (5.30–6.05)	6.25 (5.82–6.65)	6.82 (6.34–7.26)	7.39 (6.86–7.87)	8.16 (7.55–8.67)	8.75 (8.07–9.30)
7-day	3.74 (3.52–3.98)	4.42 (4.16–4.70)	5.17 (4.87–5.50)	5.76 (5.41–6.12)	6.52 (6.12–6.92)	7.11 (6.66–7.54)	7.67 (7.18–8.14)	8.23 (7.69–8.73)	8.95 (8.33–9.50)	9.49 (8.81–10.1)
10-day	4.30 (4.05–4.57)	5.07 (4.77–5.38)	5.88 (5.54–6.24)	6.50 (6.12–6.90)	7.30 (6.85–7.73)	7.90 (7.41–8.36)	8.47 (7.94–8.97)	9.02 (8.45–9.55)	9.72 (9.08–10.3)	10.2 (9.54–10.8)
20-day	5.95 (5.63–6.25)	6.97 (6.60–7.32)	7.97 (7.55–8.37)	8.71 (8.25–9.16)	9.65 (9.13–10.1)	10.3 (9.78–10.9)	11.0 (10.4–11.5)	11.6 (10.9–12.2)	12.3 (11.6–12.9)	12.9 (12.1–13.5)
30-day	7.40 (7.04–7.76)	8.64 (8.22–9.06)	9.77 (9.28–10.2)	10.6 (10.1–11.1)	11.6 (11.1–12.2)	12.4 (11.8–13.0)	13.1 (12.4–13.7)	13.7 (13.0–14.4)	14.4 (13.7–15.1)	15.0 (14.2–15.7)
45-day	9.35 (8.89–9.80)	10.9 (10.3–11.4)	12.2 (11.6–12.8)	13.1 (12.5–13.7)	14.3 (13.6–14.9)	15.1 (14.3–15.8)	15.8 (15.0–16.6)	16.5 (15.7–17.3)	17.3 (16.4–18.1)	17.8 (16.9–18.7)
60-day	11.2 (10.7–11.8)	13.0 (12.5–13.7)	14.5 (13.8–15.2)	15.5 (14.8–16.3)	16.8 (16.0–17.6)	17.7 (16.9–18.5)	18.5 (17.6–19.3)	19.2 (18.2–20.1)	20.0 (19.0–20.9)	20.5 (19.5–21.5)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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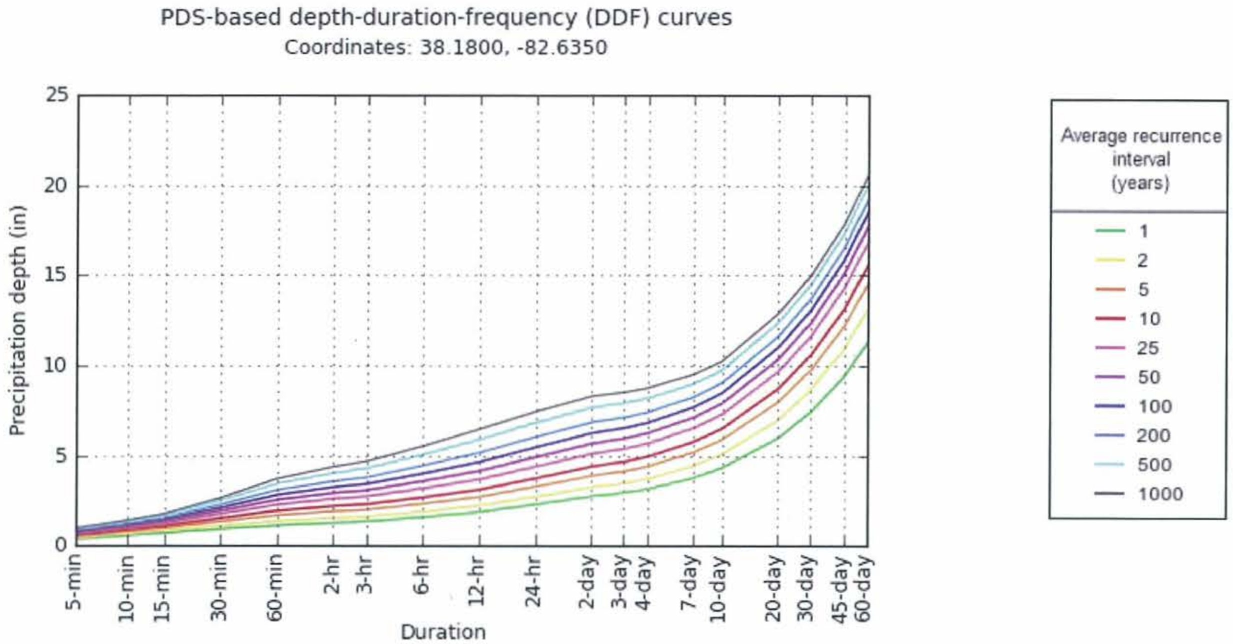
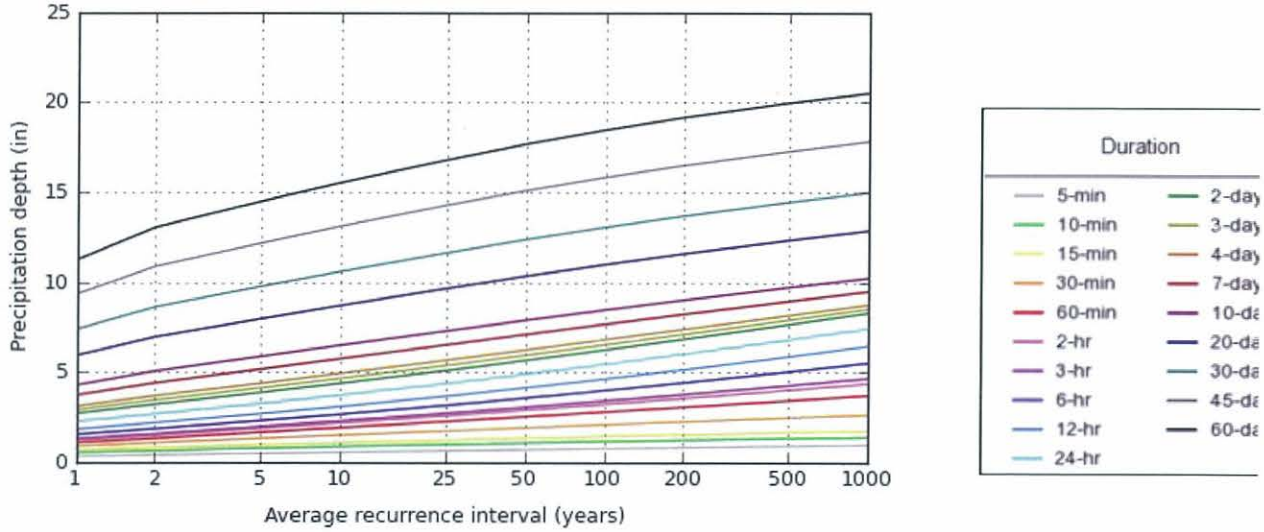
PF graphical





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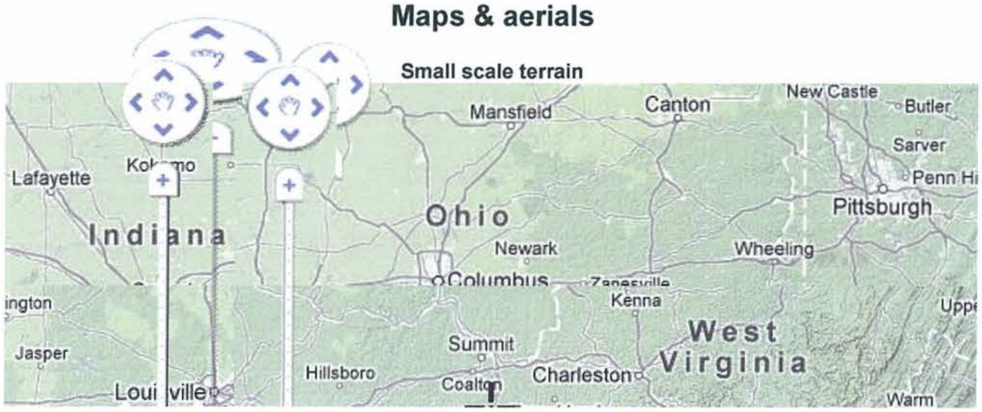
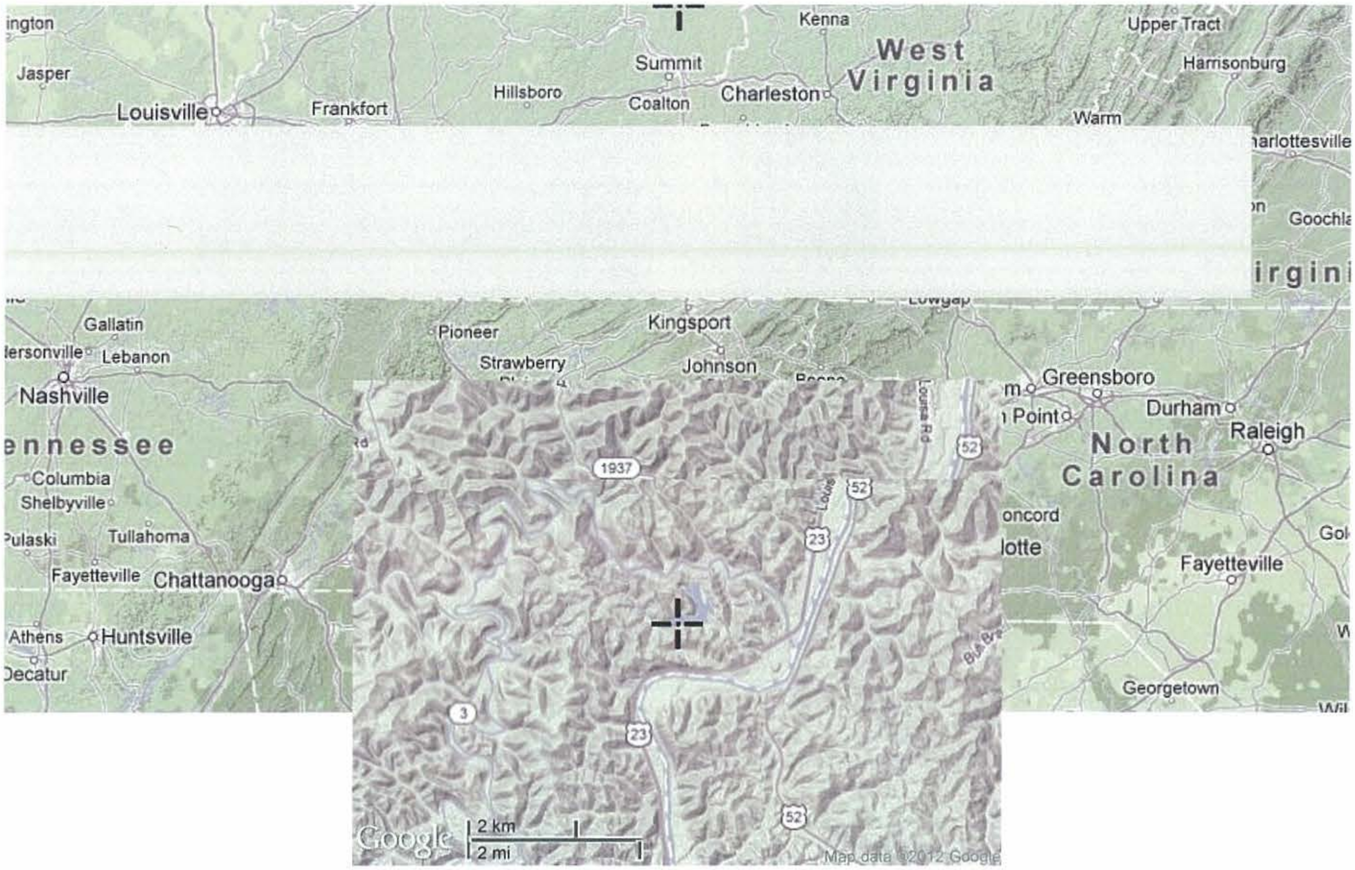
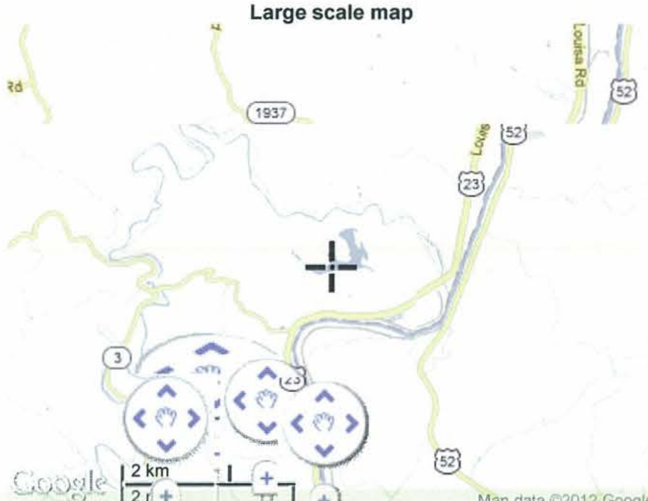
Maps & aerials

Small scale terrain

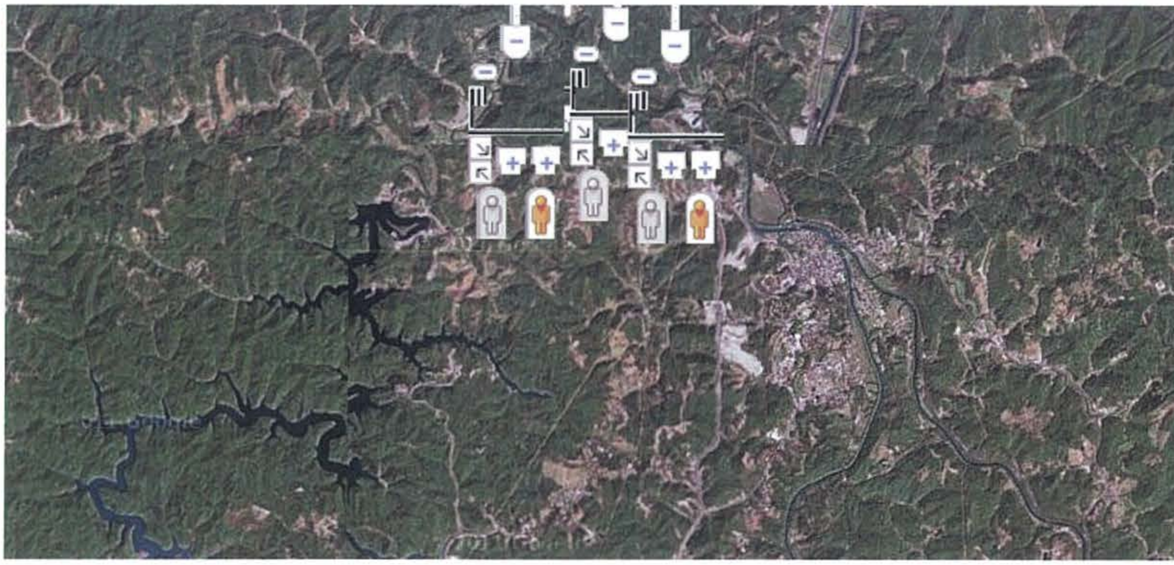




Large scale map



Large scale aerial



Department for Natural Resources and Environmental Protection
Division of Water
Engineering Memorandum No. 5

SECTION C - HYDROLOGIC CRITERIA

SECTION C - HYDROLOGIC CRITERIA

I. RUNOFF

Procedures for hydrologic design as contained in the USDA Soil Conservation Service National Engineering Handbook, Section 4 "Hydrology" will be accepted. Copies of this publication are available from the U. S. Government printing office.

The specific references for runoff determination are found in Chapter 10. All runoff volumes for design purposes will be based on Antecedent Moisture Condition II or greater. Chapter 21 contains hydrologic procedures for determining principal spillway capacities, retarding storage, and emergency spillway and freeboard hydrographs.

A. Structures in Series

For the design of a lower structure in a series, if the total drainage area above a lower structure exceeds 10 square miles and Section B-II of this memorandum applies, it is necessary to apply two sets of storms for development of both the emergency spillway and the freeboard hydrographs.

The first set of design storms will be selected for the development of the uncontrolled drainage area above a lower structure. The dimensions of the emergency spillway for a lower structure under this condition will be determined by reservoir routings of hydrographs developed for each storm.

The second set of design storms will be selected for the entire drainage area above the lower structure. Each design storm rainfall is determined by using this area in the areal adjustment of rainfall amounts. These design storm durations are determined by using the time of concentration of this area assuming no upper structures are in place. The design storm hydrographs will be routed through the emergency spillways of the upstream structures and the outflow routed to the lower structure and combined with the hydrograph for the uncontrolled area. The dimensions of the emergency spillway for a lower structure under this condition will also be determined by reservoir routings of the hydrographs developed for each storm.

The design storm imposing the most severe flow condition at the lower structure will be used.

OBJECTIVE:

Size open channel sections throughout the Saddle Dam basin.

REFERENCES:

-Bentley Systems, *FlowMaster*, V8i (SELECTseries 1), 2009.

-ESRI, *ArcGIS Desktop 10 Service Pack 5*, 2010.

-Franzini, Joseph, B., *Fluid Mechanics with Engineering Applications*, Tenth Edition, 2002.

-United States Bureau of Reclamation (USBR), *Flood Hydrology Manual*, 1989.

-URS, *Big Sandy – Frequency Event Hydrology*, 2012.

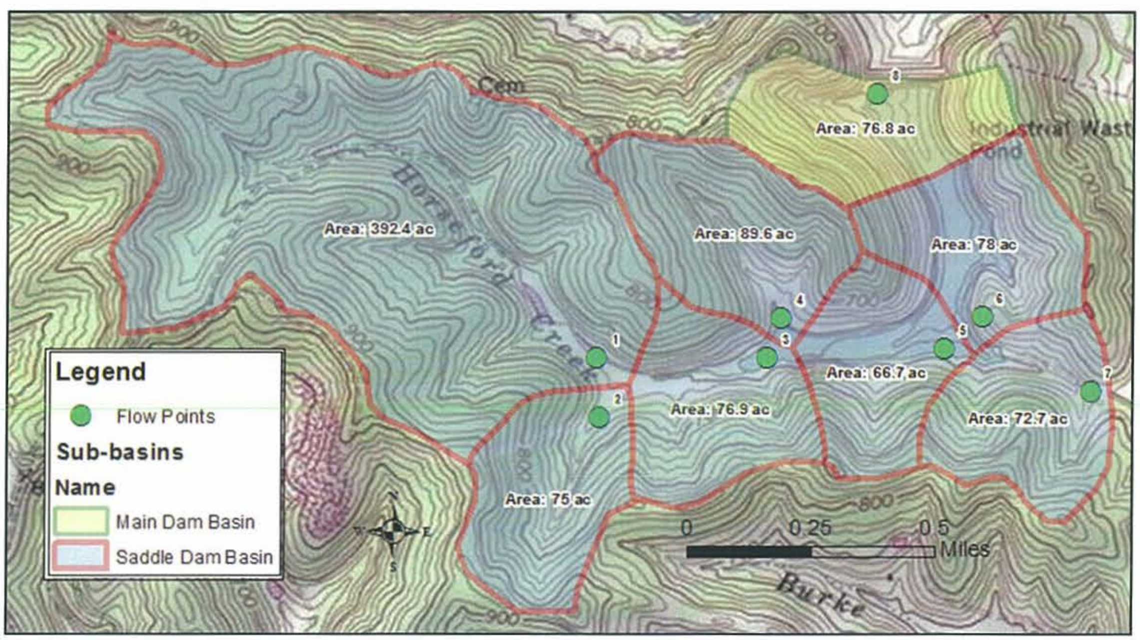
FILE LOCATION:

The contents of this calculation are located at the following network address:

T:\Projects\Glenwood\AEP Big Sandy\CALCULATIONS\Channel Sizing

APPROACH:

To size the channel sections throughout the Saddle Dam basin, the Saddle Dam basin was divided into seven sections as shown in the figure below.



Saddle Dam Basin Sections and Flow Points

The frequency storm 100 and 2 year 24 hour duration flow rates for the Saddle Dam basin were taken from the URS calculation titled *Big Sandy – Frequency Event Hydrology* (USBR 1989). These flow rates were proportionalized based on contributing area to each flow point. This was done using methodology outlined in the *Flood Hydrology Manual* (USBR 1989), which presents the equation below (also see attached reference material). See attached calculation spreadsheet for these flow rates per flow point.

$$\frac{Q_1}{Q_2} = \frac{{A_1}^{0.5}}{{A_2}^{0.5}}$$

The channel sections in each of the basin sections were then sized with a low flow section and a flood plain section. Both channel sections were designed with a one percent slope (perpendicular to the centerline of the channel) to the center of the channel. The low flow section was sized to contain the 2 year 24 hour storm event and to enable a Bulldozer (approximately 12 feet wide) to drive in the channel bottom. The flood plain section was designed to contain the 100 year 24 hour event with a minimum of 1 foot of freeboard. The total design channel depth was 5 feet. A Manning's roughness coefficient (n) of 0.033 was assumed for the grass lined channel (Franzini 2002, see attached reference material). The channel slope throughout the basin was assumed to be 0.5 percent. FlowMaster (Bentley Systems 2009) was used to size each channel section. For flow points 2, 4, and 6, single channel section was sized. The maximum velocity of the channel sections was 5.3 ft/s. See attached for the FlowMaster output and an Excel spreadsheet showing the channel cross-sections.

RESULTS:

Channel sections were designed throughout the Saddle Dam basin to contain the 100 year frequency 24 hour duration event in the flood plain, and the 2 year frequency 24 hour event in the low flow channel. A total channel depth of 5 feet with a minimum of one foot of freeboard was used for all cross-sections. See attached Excel spreadsheet for final channel cross-section parameters.

Big Sandy
Proportionalized Flows

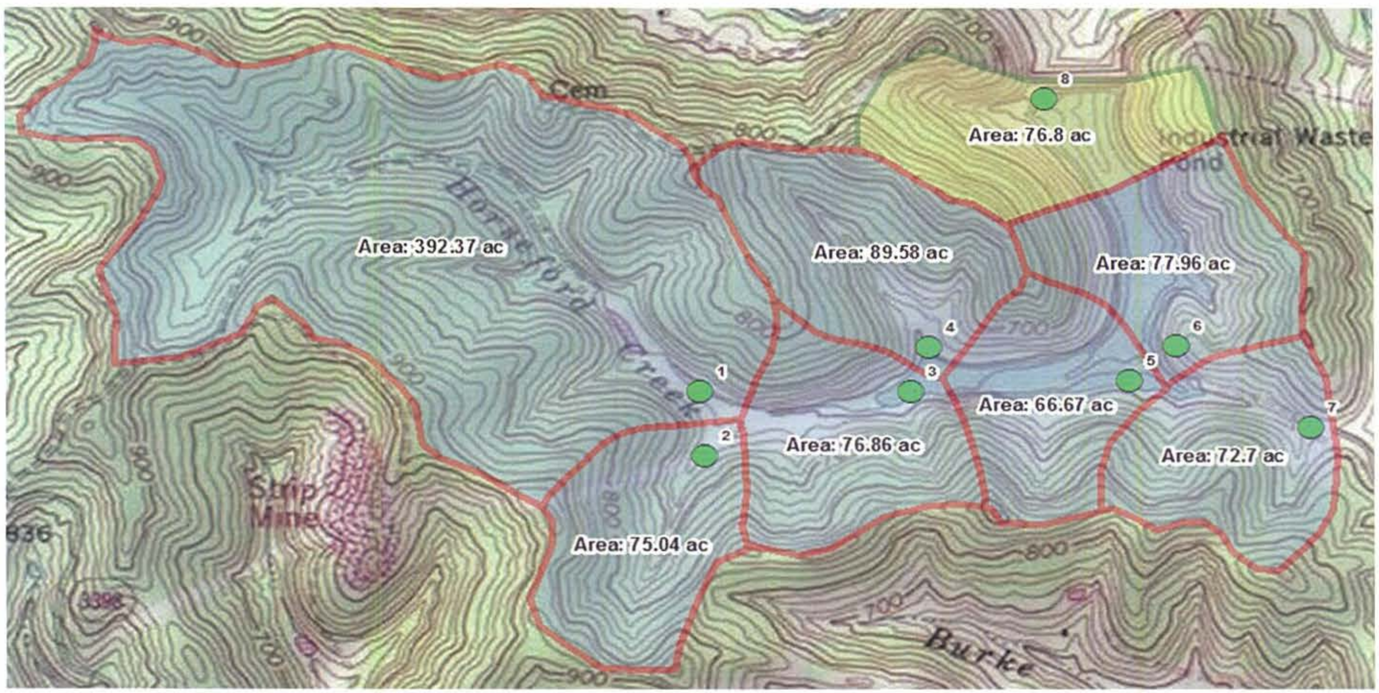
	Saddle Dam	Main Dam	
100yr 24hr Inflow	844	216	cfs

Flow Point	Contributing Area	Discharge[1]	Floodplain Width[2]	Average Velocity	Normal Depth	Freeboard
-	ac	cfs	ft	ft/s	ft	ft
1	392.4	573	62.0	5.2	3.7	1.3
2	75.0	251	29.6	5.3	3.0	2.0
3	544.3	675	74.0	5.1	3.8	1.3
4	89.6	274	29.6	5.3	3.0	2.0
5	700.5	766	84.5	5.1	3.8	1.2
6	78.0	255	29.6	5.3	3.0	2.0
7	851.2	844	95.0	5.0	3.8	1.2
8	76.8	216	No Channel	No Channel	No Channel	No Channel

Equation Reference: USBR, *Flood Hydrology Manual*, First Eddition, 1989.

$$\frac{Q_1}{Q_2} = \frac{A_1^{0.5}}{A_2^{0.5}}$$

[1]Discharge based on 100 year 24 hour storm event.
[2]The floodplain width is defined as the channel top width.



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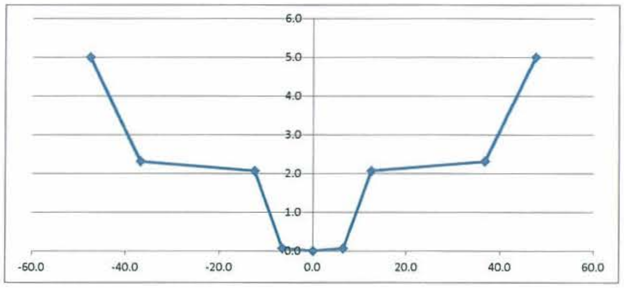
Big Sandy
Saddle Dam Channel Sections

User Input

Flow Point 7

100 YR 24 HR Q	844.0	cfs
Bottom with of low flow	13.0	ft
Top width of low flow	25.0	ft
Bottom width of high flow	73.5	ft
Top width of high flow	95.0	ft

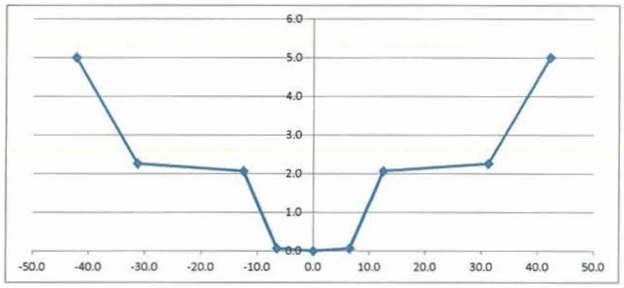
x	y
ft	ft
-47.5	5.0
-36.8	2.3
-12.5	2.1
-6.5	0.1
0.0	0.0
6.5	0.1
12.5	2.1
36.8	2.3
47.5	5.0



Flow Point 5

100 YR 24 HR Q	765.7	cfs
Bottom with of low flow	13.0	ft
Top width of low flow	25.0	ft
Bottom width of high flow	62.5	ft
Top width of high flow	84.5	ft

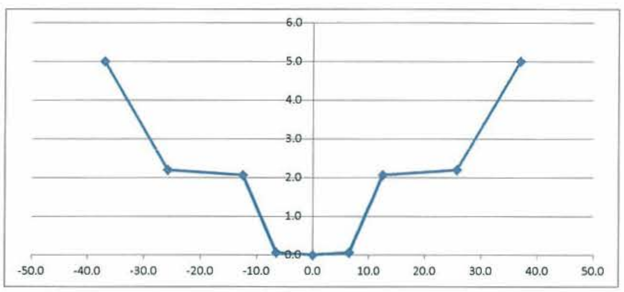
x	y
ft	ft
-42.2	5.0
-31.3	2.3
-12.5	2.1
-6.5	0.1
0.0	0.0
6.5	0.1
12.5	2.1
31.3	2.3
42.2	5.0



Flow Point 3

100 YR 24 HR Q	674.9	cfs
Bottom with of low flow	13.0	ft
Top width of low flow	25.0	ft
Bottom width of high flow	51.5	ft
Top width of high flow	73.9	ft

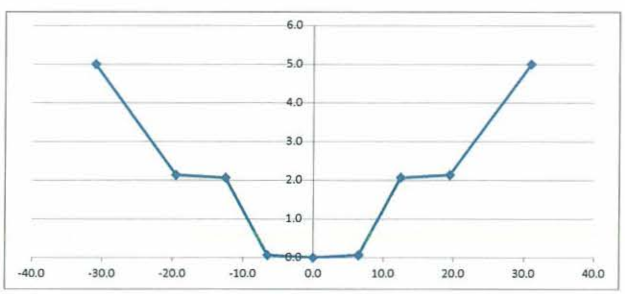
x	y
ft	ft
-37.0	5.0
-25.8	2.2
-12.5	2.1
-6.5	0.1
0.0	0.0
6.5	0.1
12.5	2.1
25.8	2.2
37.0	5.0



Flow Point 1

100 YR 24 HR Q	573.0	cfs
Bottom with of low flow	13.0	ft
Top width of low flow	25.0	ft
Bottom width of high flow	39.0	ft
Top width of high flow	61.9	ft

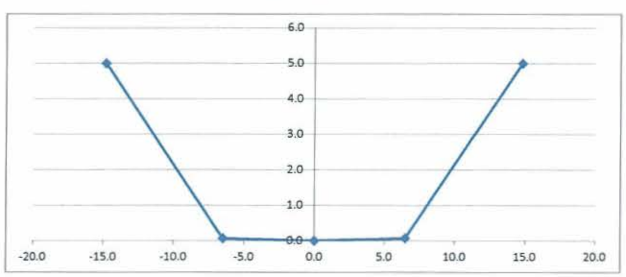
x	y
ft	ft
-31.0	5.0
-19.5	2.1
-12.5	2.1
-6.5	0.1
0.0	0.0
6.5	0.1
12.5	2.1
19.5	2.1
31.0	5.0



Flow Points 2, 4, 6

100 YR 24 HR Q	273.8	cfs
Bottom with of low flow	13.0	ft
Depth	5.0	ft
Top width of low flow	29.6	ft

x	y
ft	ft
-14.8	5.0
-6.5	0.1
0.0	0.0
6.5	0.1
14.8	5.0



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Cross Section for Channel X-Sec 3

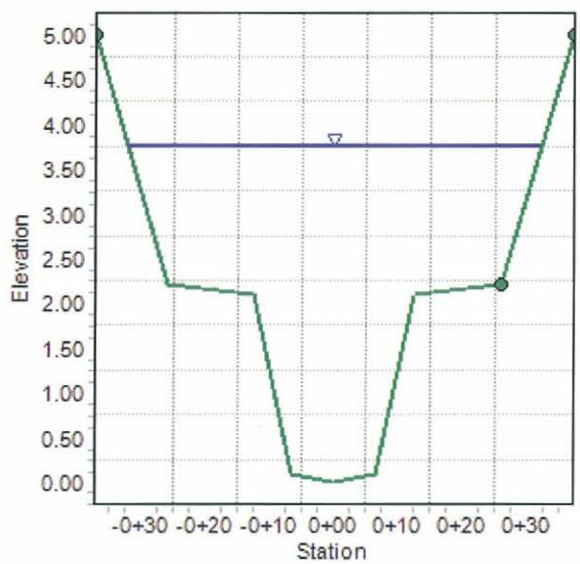
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Normal Depth	3.75	ft
Discharge	674.90	ft^3/s

Cross Section Image



Worksheet for Channel X-Sec 3

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Discharge	674.90	ft^3/s

Section Definitions

Station (ft)	Elevation (ft)
-0+37	5.00
-0+26	2.20
-0+13	2.10
-0+07	0.10
0+00	0.00
0+07	0.10
0+13	2.10
0+26	2.20
0+37	5.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-0+37, 5.00)	(0+26, 2.20)	0.033
(0+26, 2.20)	(0+37, 5.00)	0.033

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	3.75	ft
Elevation Range	0.00 to 5.00 ft	
Flow Area	132.14	ft^2

Worksheet for Channel X-Sec 3

Results

Wetted Perimeter		65.04	ft
Hydraulic Radius		2.03	ft
Top Width		64.01	ft
Normal Depth		3.75	ft
Critical Depth		3.13	ft
Critical Slope		0.01385	ft/ft
Velocity		5.11	ft/s
Velocity Head		0.41	ft
Specific Energy		4.16	ft
Froude Number		0.63	
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	3.75	ft
Critical Depth	3.13	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.01385	ft/ft

Cross Section for Channel X-Sec 7 (low flow)

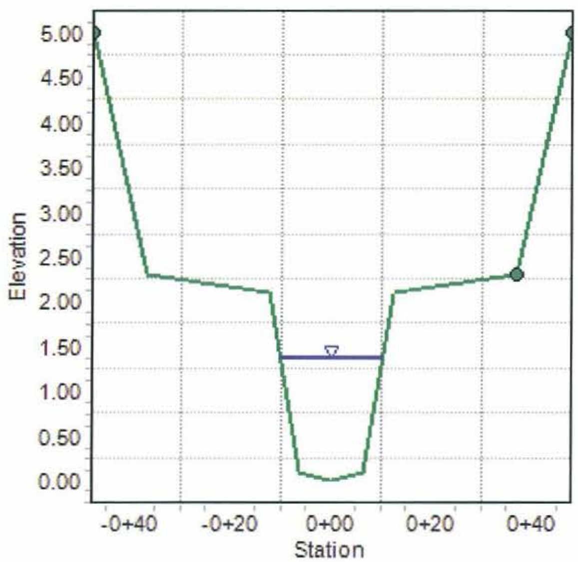
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Normal Depth	1.37	ft
Discharge	72.00	ft³/s

Cross Section Image



Worksheet for Channel X-Sec 7 (low flow)

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Discharge	72.00	ft^3/s

Section Definitions

Station (ft)	Elevation (ft)
-0+48	5.00
-0+37	2.30
-0+13	2.10
-0+07	0.10
0+00	0.00
0+07	0.10
0+13	2.10
0+37	2.30
0+48	5.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-0+48, 5.00)	(0+37, 2.30)	0.033
(0+37, 2.30)	(0+48, 5.00)	0.033

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	1.37	ft
Elevation Range	0.00 to 5.00 ft	
Flow Area	21.97	ft^2

Worksheet for Channel X-Sec 7 (low flow)

Results

Wetted Perimeter	21.02	ft
Hydraulic Radius	1.04	ft
Top Width	20.61	ft
Normal Depth	1.37	ft
Critical Depth	0.98	ft
Critical Slope	0.01756	ft/ft
Velocity	3.28	ft/s
Velocity Head	0.17	ft
Specific Energy	1.54	ft
Froude Number	0.56	
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	1.37	ft
Critical Depth	0.98	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.01756	ft/ft

Cross Section for Channel X-Sec 7

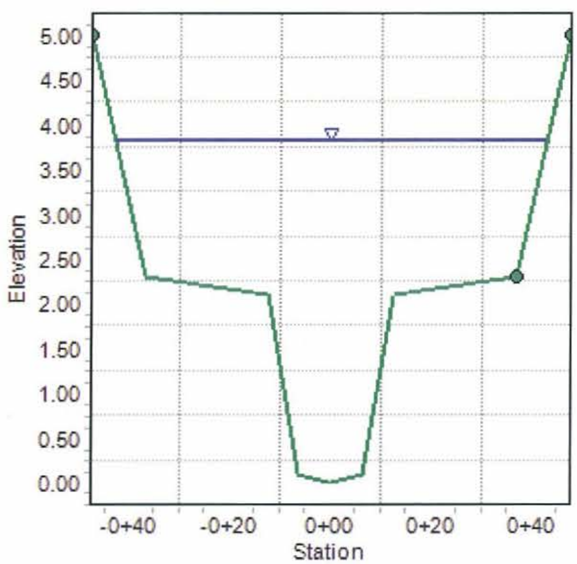
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Normal Depth	3.82	ft
Discharge	844.00	ft³/s

Cross Section Image



Worksheet for Channel X-Sec 7

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Discharge	844.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
-0+48	5.00
-0+37	2.30
-0+13	2.10
-0+07	0.10
0+00	0.00
0+07	0.10
0+13	2.10
0+37	2.30
0+48	5.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-0+48, 5.00)	(0+37, 2.30)	0.033
(0+37, 2.30)	(0+48, 5.00)	0.033

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	3.82	ft
Elevation Range	0.00 to 5.00 ft	
Flow Area	169.50	ft²

Worksheet for Channel X-Sec 7

Results

Wetted Perimeter	86.67	ft
Hydraulic Radius	1.96	ft
Top Width	85.64	ft
Normal Depth	3.82	ft
Critical Depth	3.24	ft
Critical Slope	0.01407	ft/ft
Velocity	4.98	ft/s
Velocity Head	0.39	ft
Specific Energy	4.20	ft
Froude Number	0.62	
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	3.82	ft
Critical Depth	3.24	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.01407	ft/ft

Cross Section for Channel X-Sec 5

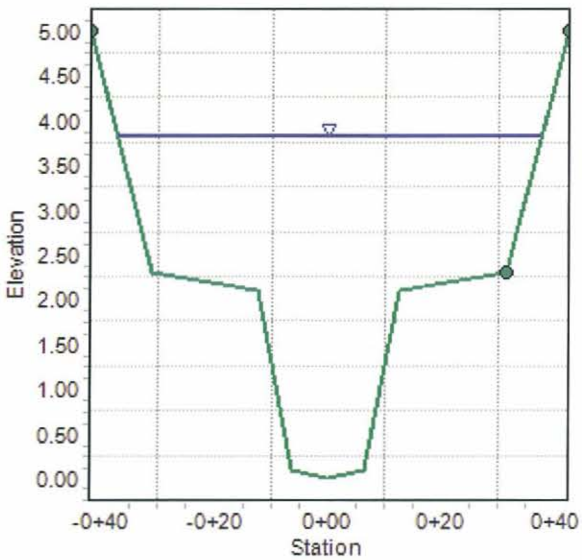
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Normal Depth	3.82	ft
Discharge	765.70	ft³/s

Cross Section Image



Worksheet for Channel X-Sec 5

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Discharge	765.70	ft³/s
Section Definitions		

Station (ft)	Elevation (ft)
-0+42	5.00
-0+31	2.30
-0+13	2.10
-0+07	0.10
0+00	0.00
0+07	0.10
0+13	2.10
0+31	2.30
0+42	5.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-0+42, 5.00)	(0+31, 2.30)	0.033
(0+31, 2.30)	(0+42, 5.00)	0.033

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth		3.82	ft
Elevation Range	0.00 to 5.00 ft		
Flow Area		151.59	ft²

Worksheet for Channel X-Sec 5

Results

Wetted Perimeter		75.86	ft
Hydraulic Radius		2.00	ft
Top Width		74.84	ft
Normal Depth		3.82	ft
Critical Depth		3.22	ft
Critical Slope		0.01395	ft/ft
Velocity		5.05	ft/s
Velocity Head		0.40	ft
Specific Energy		4.21	ft
Froude Number		0.63	
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	3.82	ft
Critical Depth	3.22	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.01395	ft/ft

Cross Section for Channel X-Sec 1

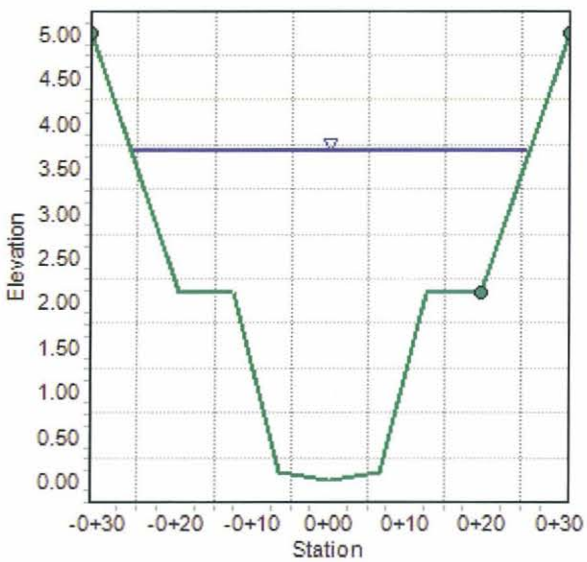
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Normal Depth	3.68	ft
Discharge	573.00	ft³/s

Cross Section Image



Worksheet for Channel X-Sec 1

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Discharge	573.00	ft^3/s

Section Definitions

Station (ft)	Elevation (ft)
-0+31	5.00
-0+20	2.10
-0+13	2.10
-0+07	0.10
0+00	0.00
0+07	0.10
0+13	2.10
0+20	2.10
0+31	5.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-0+31, 5.00)	(0+20, 2.10)	0.033
(0+20, 2.10)	(0+31, 5.00)	0.033

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	3.68	ft
Elevation Range	0.00 to 5.00 ft	
Flow Area	109.98	ft^2

Worksheet for Channel X-Sec 1

Results

Wetted Perimeter	52.54	ft
Hydraulic Radius	2.09	ft
Top Width	51.50	ft
Normal Depth	3.68	ft
Critical Depth	3.02	ft
Critical Slope	0.01368	ft/ft
Velocity	5.21	ft/s
Velocity Head	0.42	ft
Specific Energy	4.10	ft
Froude Number	0.63	
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	3.68	ft
Critical Depth	3.02	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.01368	ft/ft

Cross Section for Channel X-Sec 2, 4, 6

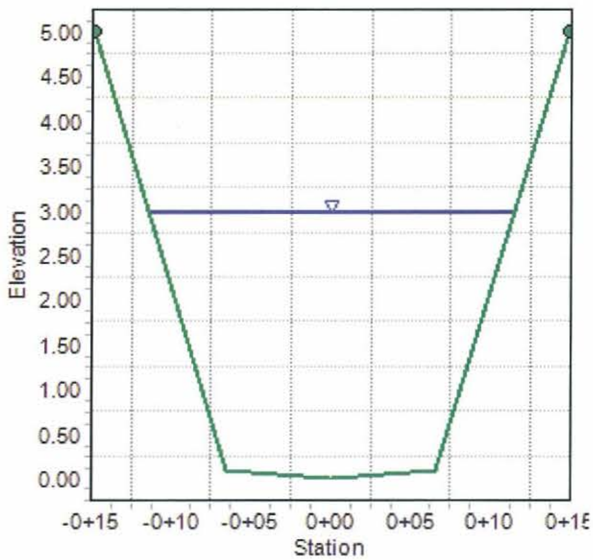
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Normal Depth	2.97	ft
Discharge	273.80	ft^3/s

Cross Section Image



Worksheet for Channel X-Sec 2, 4, 6

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00500	ft/ft
Discharge	273.80	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
-0+15	5.00
-0+07	0.10
0+00	0.00
0+07	0.10
0+15	5.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-0+15, 5.00)	(0+15, 5.00)	0.033

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth	2.97	ft
Elevation Range	0.00 to 5.00 ft	
Flow Area	51.86	ft²
Wetted Perimeter	24.28	ft
Hydraulic Radius	2.14	ft
Top Width	22.71	ft
Normal Depth	2.97	ft
Critical Depth	2.23	ft

Worksheet for Channel X-Sec 2, 4, 6

Results

Critical Slope		0.01410	ft/ft
Velocity		5.28	ft/s
Velocity Head		0.43	ft
Specific Energy		3.40	ft
Froude Number		0.62	
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	2.97	ft
Critical Depth	2.23	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.01410	ft/ft

Flood Hydrology Manual

A Water Resources Technical Publication

by
Arthur G. Cudworth, Jr.

Surface Water Branch
Earth Sciences Division

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United States Department of the Interior
Bureau of Reclamation
Denver Office

vary considerably between unit times. This type of conflict must be resolved in a reasonable manner. The most common approach would be to achieve some smoothing of the skew coefficients with respect to unit time. This type of problem is usually only found at the longer unit times, and the inconsistency usually amounts to only an insignificant volume.

(b) Fixed Interval Analysis.—One source of potential problems in the construction and use of balanced hydrographs is what is often called "fixed interval analysis." For example, in the construction of the balanced hydrograph, the peak 1-day volume is used. This value is determined from daily flow values, not peak 24-hour values. Note that peak daily values are constrained to start at midnight, thus the term "fixed interval," while the peak 24-hour values start at any time of day. A peak 24-hour value for 1 year will always exceed the peak daily value. When constructing the balanced hydrograph, the peak daily flow value is usually used as being the same as the peak 24-hour value. As a result, the balanced hydrograph has slightly less volume near the peak. This problem is considered to be of minor concern, but some awareness is justified.

A more serious fixed interval problem exists if monthly data are taken to be 30-day volume data. Again, the use of fixed interval data will result in low estimates, and care should be taken to quantify the magnitude of this problem. The decision as to whether the fixed interval data is adequate will depend on how critical the volume frequency data is to the study at hand.

7.9 Probability Relationships for Ungauged Areas

Usually, the site where the frequency analysis is needed is not a site of recorded flow data, and the site is said to be an "ungauged area." Three basic approaches are used to estimate a frequency curve for such an ungauged site: (1) transposition of frequency data, (2) regional flood frequency, and (3) use of synthetic data.

(a) Transposition of Frequency Data.—The transfer of information from one site to another is required for any ungauged area. A basic relationship found to work well is that the ratio of flows at two locations are assumed equal to the ratio of the square root of the drainage areas, or:

$$\frac{Q_1}{Q_2} = \frac{A_1^{0.5}}{A_2^{0.5}} \qquad (5)$$

This square root relationship has been found to be fairly accurate for instantaneous peaks and for the short duration volume frequency values. For longer duration volumes of about 60 days or more, a higher exponential power of the ratio of the drainage areas may be more applicable, with the power increasing from 0.5 to 0.8 as the unit time is increased.

For annual runoff, this power may approach 1.0. It should be noted that this approach does not take into account any differences in basin factors other than drainage area alone. In effect, the LP-III frequency curve being transposed is only changed in terms of the mean; the standard deviation and coefficient of skew are not altered.

It is preferable to transpose information from one site to another on the same stream rather than from one stream to another. Lacking this preference, it is preferable to transfer information between basins that are hydrologically similar. Small changes in basin size are also preferable. Also, some attention should be given to the similarities and differences between the sources of floods within the basins.

(b) Regional Flood Frequency.—There are three basic approaches to regional flood frequency: (1) average parameter approach, (2) specific frequency flood approach, and (3) index flood approach. The depth and scope of these three approaches vary considerably; however, similar results are obtained from all three.

The average parameter approach uses data at many similar sites in a homogeneous region to estimate an average value for a parameter. In many applications, the parameter is not really an average but rather a function of basin factors. In effect, the value for the parameter is an average value for that basin's size and characteristics. The most common average parameter approach used is to assume that the mean value parameter (logarithms) varies with the drainage area, while the standard deviation and skew coefficient do not. Generally, the log mean of the flows is plotted against drainage area and a graphical relationship developed. The next most common type of parameter approach would be to relate the mean not only to area but also to other factors such as location, annual precipitation, basin elevation, and cover. Another more complex and most often not justifiable type of average parameter approach is to also relate the standard deviation to basin factors. The coefficient of skew could also be related to other factors (generalized skew) but, for an ungauged area, this refinement often cannot be justified.

The specific frequency flood approach begins by analyzing the flood frequency at many stations in the region. Then, a relationship is developed that relates the values for a single return period, such as the 100-year values, to the basin areas and other basin factors. Obviously, this approach produces results similar to those obtained using the average parameter approach.

The index flood approach usually requires the use of concurrent data from several stations, with the data from each station being scaled using the mean annual flood or some similar value. In some cases, the drainage area (square root) might be used to scale the data. The resulting scaled data are then combined as if all the data came from a single site. When

Fluid Mechanics

with Engineering Applications

Tenth Edition

E. John Finnemore

Joseph B. Franzini

where 1.486 is the cube root of 3.28, the number of feet in a meter. Despite the dimensional difficulties of the Manning formula, which have long plagued 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: $$Q(\text{cfs}) = \frac{1.486}{n} A R_h^{2/3} S_0^{1/2} \qquad (10.8a)$$

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

TABLE 10.1 **Values of *n* in Manning's formula**

	n	
Nature of surface	**Min**	**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

OBJECTIVE:

Route the freeboard (PMP) and emergency spillway hydrographs through the Main Dam and Saddle Dam basins at Big Sandy.

REFERENCES:

-URS, *Big Sandy – Frequency Event Hydrology*, 2012.

-U.S. Army Corps of Engineers (USACE), *Hydrologic Modeling System (HEC-HMS)*, Version: 3.5, Build: 1417, August 10, 2010.

-Kentucky Division of Water (KDOW), Department of Natural Resources and Environmental Protection, *Rainfall Frequency Values for Kentucky*, Engineering Memorandum No. 2, April 30, 1971, revised June 1, 1979.

- Kentucky Division of Water (KDOW), Department of Natural Resources and Environmental Protection, *Design Criteria for Dams and Associated Structures*, Engineering Memorandum No. 5, June 1999.

-Geo/Environmental Associates, Inc., *Hydrologic Analysis, Big Sandy Plant – Horseford Creek Fly Ash Dam*, Lawrence County, Kentucky, December 15, 2006.

FILE LOCATION:

The contents of this calculation are located at the following network address:

T:\Projects\Glenwood\AEP Big Sandy\CALCULATIONS\PMP Routing

APPROACH:

A HEC-HMS (USACE 2010) model was built to route the PMP and emergency spillway hydrographs through the Main Dam and Saddle Dam basins. The model was developed following the steps presented below.

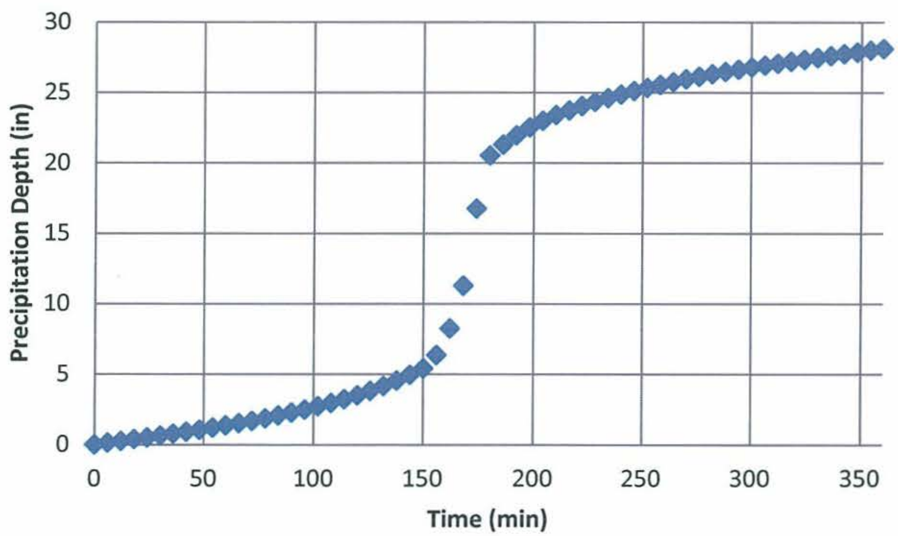
Step 1: Define the freeboard (PMP) and emergency spillway design hydrographs.
Step 2: Define the attributes (e.g., loss parameters, areas) of the Main Dam and Saddle Dam basins.
Step 3: Estimate the stage-area relationships for the Main Dam and Saddle Dam basins.
Step 4: Define the spillway characteristics for the Main Dam and Saddle Dam basins.
Step 5: Run the model.

<u>Step 1: Define the freeboard (PMP) and emergency spillway design hydrographs.</u>

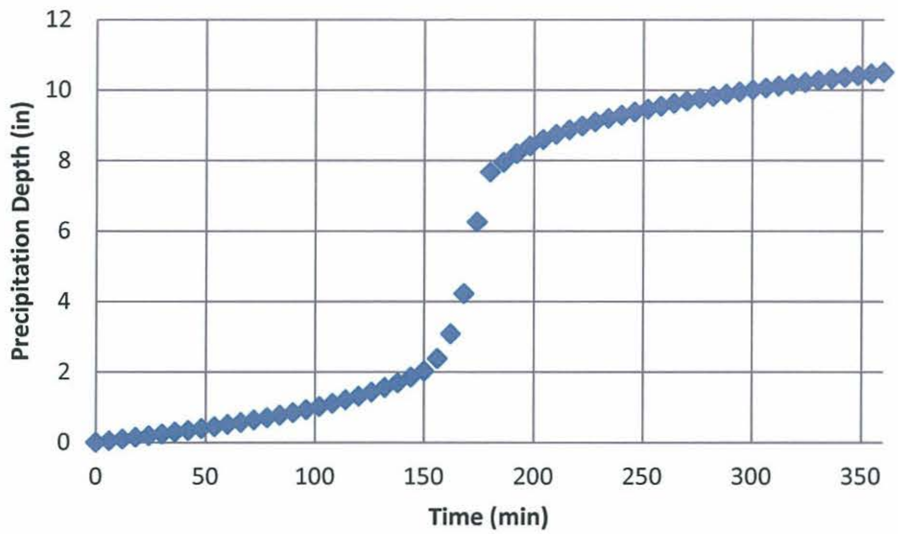
The 6 hour rainfall depth was selected from *Rainfall Frequency Values for Kentucky* (KDOW 1979). The 100 year (P_{100}) and PMP rainfall values for Lawrence County are 4.3 and 28.1 inches. In accordance with *Design Criteria for Dams and Associated Structures* (KDOW 1999), the emergency spillway and freeboard design rainfall values was determined for a high hazard facility (Class C). The freeboard rainfall depth is simply the PMP rainfall depth. The emergency spillway rainfall depth was determined with the equation shown below yielding 10.5 inches.

$$P_c = P_{100} + 0.40(PMP - P_{100})$$

The *Design Criteria for Dams and Associated Structures* (KDOW 1999) guidelines specify the use of SCS methodology to development 6 hour duration hydrographs for both the freeboard (PMP) and emergency spillway hydrographs. The SCS S-curve ordinates were taken directly from a previous hydrologic study report of this facility (Geo/Environmental Associates 2006). The resulting S-curves after applying the emergency spillway and freeboard (PMP) rainfall values are shown in the figures below. See attached for the tabular data. The S-Curves were input into the HEC-HMS model as precipitation gauges.



Freeboard (PMP) S-Curve – 6 Hour Duration – SCS Distribution



Emergency Spillway S-Curve – 6 Hour Duration – SCS Distribution

Step 2: Define the attributes (e.g., loss parameters, areas) of the Main Dam and Saddle Dam basins.

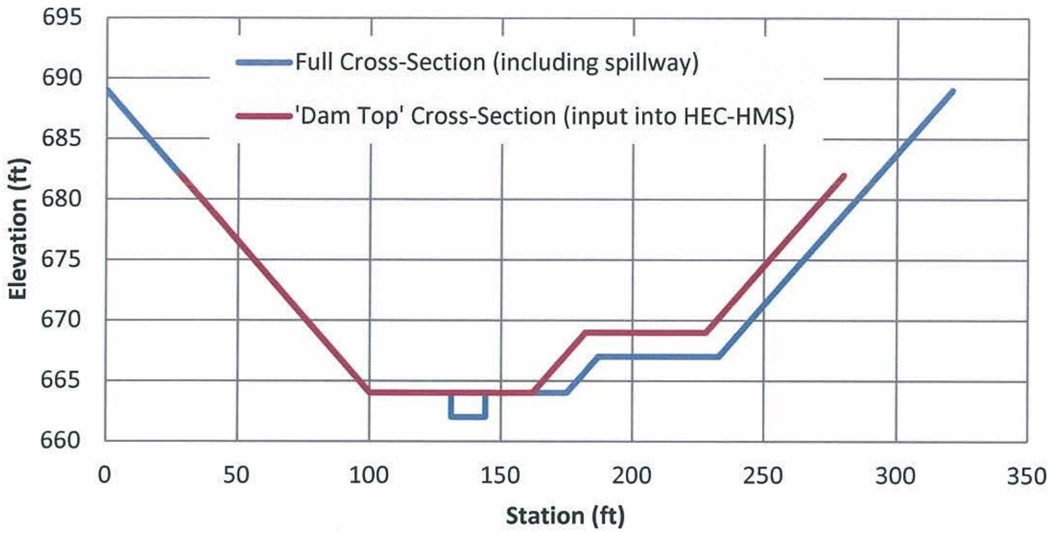
The SCS curve number and SCS unity hydrograph options were utilized in the HEC-HMS model. The Main Dam and Saddle basin attributes required were the basin area, initial abstraction, curve number, and lag time. For these values see the URS calculation packet titled Big Sandy – Frequency Event Hydrology (URS 2012).

Step 3: Estimate the stage-area relationships for the Main Dam and Saddle Dam basins.

The stage-area relationships were developed for each basin using the proposed (per grading plan dated 10-22-12). This data was put into the HEC-HMS model (see attached for the tabular data).

Step 4: Define the spillway characteristics for the Main Dam and Saddle Dam basins.

The Main Dam spillway consists of a 10 foot wide broad crested weir which was input into the HEC-HMS as a broad crested weir. The Saddle Dam spillway consists of a low flow section, a trapezoidal section designed to contain the emergency spillway hydrograph, and a trapezoidal section design to contain the freeboard (PMP) hydrograph. The low flow section has a crest elevation of 662 feet and a crest length of 13 feet which was modeled in HEC-HMS as a broad crested weir. The remaining section was modeled as a 'dam top' in HEC-HMS; the cross-section is shown in the figure below. Because all spillways resemble broad-crested weirs, a weir coefficient of 2.6 was used.



Saddle Dam Spillway (full cross-section and section modeled as a 'dam top' in HEC-HMS)

RESULTS:

The results from the HEC-HMS model are presented in the table below.

Results Table

Parameter	Emergency Spillway: Saddle Dam	Emergency Spillway: Main Dam	Freeboard (PMP): Saddle Dam	Freeboard (PMP): Main Dam
Inflow (cfs)	3,319	833	12,418	2,825
Outflow (cfs)	3,289	250	12,167	862
Peak Elevation (ft)	669.0	649.5	674.3	655.3
Depth (ft)*	7.0	4.5	12.3	10.3

*Relative to spillway crest elevation (Main Dam = 645 feet / Saddle Dam = 662 feet).

ATTACHED:

-Reference material (KDOW 1979 & 1999, Geo/Environmental Associates 2006).

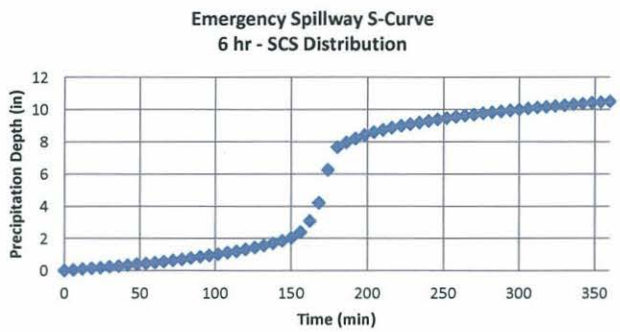
Emergency Spillway S-Graph

SCS 6 Hour Distribution Taken From: Geo/Environmental Associates Report Titled Hydrologic Analyses Big Sandy Plant - Horseford Creek Fly Ash Dam, 2006.

100 Year 6 Hour Precip:	4.3	in
PMP 6 Hour Precip:	28.1	in
Emergency Spillway Design Precip:	10.5	in

<--- $P_C = P_{100} + 0.26 x (PMP - P_{100})$

Time	S-Graph (SCS Distribution) Factor	Precipitation
min	-	in
0	0.00000	0.00000
6	0.00453	0.04757
12	0.00906	0.09513
18	0.01359	0.14270
24	0.01812	0.19026
30	0.02265	0.23783
36	0.02729	0.28655
42	0.03216	0.33768
48	0.03725	0.39113
54	0.04250	0.44625
60	0.04812	0.50526
66	0.05396	0.56658
72	0.06013	0.63137
78	0.06664	0.69972
84	0.07349	0.77165
90	0.08068	0.84714
96	0.08832	0.92736
102	0.09653	1.01357
108	0.10531	1.10576
114	0.11460	1.20330
120	0.12456	1.30788
126	0.13543	1.42202
132	0.14766	1.55043
138	0.16125	1.69313
144	0.17619	1.85000
150	0.19250	2.02125
156	0.22624	2.37552
162	0.29350	3.08175
168	0.40168	4.21764
174	0.59570	6.25485
180	0.73036	7.66878
186	0.75720	7.95060
192	0.78081	8.19851
198	0.80119	8.41250
204	0.81834	8.59257
210	0.83227	8.73884
216	0.84422	8.86431
222	0.85543	8.98202
228	0.86590	9.09195
234	0.87560	9.19380
240	0.88464	9.28872
246	0.89308	9.37734
252	0.90112	9.46176
258	0.90876	9.54198
264	0.91601	9.61811
270	0.92286	9.69003
276	0.92937	9.75839
282	0.93560	9.82380
288	0.94154	9.88617
294	0.94730	9.94665
300	0.95258	10.00209
306	0.95778	10.05669
312	0.96286	10.11003
318	0.96786	10.16253
324	0.97274	10.21377
330	0.97754	10.26417
336	0.98222	10.31331
342	0.98681	10.36151
348	0.99131	10.40876
354	0.99570	10.45485
360	1.00000	10.50000

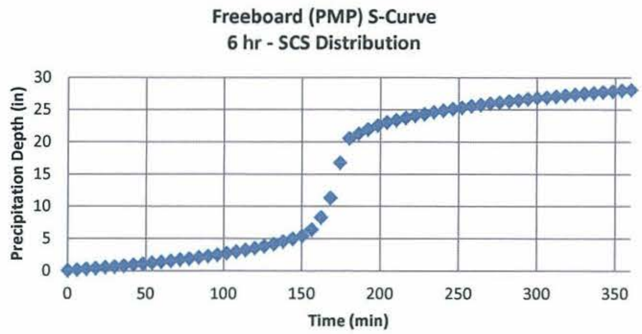


Freeboard (PMP) S-Graph

SCS 6 Hour Distribution Taken From: Geo/Environmental Associates Report Titled Hydrologic Analyses Big Sandy Plant - Horseford Creek Fly Ash Dam, 2006.

PMP 6 Hour Precip: 28.1 in

Time	S-Graph (SCS Distribution) Factor	Precipitation
min	-	in
0	0.00000	0.00000
6	0.00453	0.12729
12	0.00906	0.25459
18	0.01359	0.38188
24	0.01812	0.50917
30	0.02265	0.63647
36	0.02729	0.76685
42	0.03216	0.90370
48	0.03725	1.04673
54	0.04250	1.19425
60	0.04812	1.35217
66	0.05396	1.51628
72	0.06013	1.68965
78	0.06664	1.87258
84	0.07349	2.06507
90	0.08068	2.26711
96	0.08832	2.48179
102	0.09653	2.71249
108	0.10531	2.95921
114	0.11460	3.22026
120	0.12456	3.50014
126	0.13543	3.80558
132	0.14766	4.14925
138	0.16125	4.53113
144	0.17619	4.95094
150	0.19250	5.40925
156	0.22624	6.35734
162	0.29350	8.24735
168	0.40168	11.28721
174	0.59570	16.73917
180	0.73036	20.52312
186	0.75720	21.27732
192	0.78081	21.94076
198	0.80119	22.51344
204	0.81834	22.99535
210	0.83227	23.38679
216	0.84422	23.72258
222	0.85543	24.03758
228	0.86590	24.33179
234	0.87560	24.60436
240	0.88464	24.85838
246	0.89308	25.09555
252	0.90112	25.32147
258	0.90876	25.53616
264	0.91601	25.73988
270	0.92286	25.93237
276	0.92937	26.11530
282	0.93560	26.29036
288	0.94154	26.45727
294	0.94730	26.61913
300	0.95258	26.76750
306	0.95778	26.91362
312	0.96286	27.05637
318	0.96786	27.19687
324	0.97274	27.33399
330	0.97754	27.46887
336	0.98222	27.60038
342	0.98681	27.72936
348	0.99131	27.85581
354	0.99570	27.97917
360	1.00000	28.10000



Big Sandy - Saddle Dam Basin
Stage-Area
Per Grading Plan Dated 10-22-12

Elevation	Area	
FT	FT^2	AC
662	2220.16	0.05
664	16774.02	0.39
666	49721.97	1.14
668	101985.5	2.34
670	212821.7	4.89
672	368399.3	8.46
674	582773	13.38
676	891760.8	20.47
678	1262457	28.98
680	1750762	40.19
682	2293379	52.65

Big Sandy - Main Dam Basin
Stage-Area
Per Grading Plan Dated 10-22-12

Elevation	Area	
FT	FT^2	AC
645	0	0.00
646	175369.2	4.03
648	234575	5.39
650	302133.2	6.94
652	372387.3	8.55
654	455182.5	10.45
656	533230.3	12.24
658	610759.4	14.02
662	758486	17.41
664	812725.1	18.66
666	864748.4	19.85
668	910912.9	20.91
670	958934.3	22.01
672	990494.4	22.74
674	1023310	23.49
675	1041697	23.91
676	1052351	24.16
678	1073684	24.65
682	1115194	25.60
684	1134996	26.06
685	1144821	26.28
686	1155347	26.52
688	1175911	27.00
690	1197466	27.49

HYDROLOGIC ANALYSES
BIG SANDY PLANT - HORSEFORD CREEK FLY ASH DAM
LAWRENCE COUNTY, KENTUCKY

Prepared For:

AEP Service Corporation
Geotechnical Engineering Group
1 Riverside Plaza
Columbus, OH 43215-2373

Prepared By:

Geo/Environmental Associates, Inc.
3502 Overlook Circle
Knoxville, TN 37909

GA Project No. 06-365
December 15, 2006

GA

feet. Additionally, a saddle dam is constructed toward the rear of the impoundment. The current pool level in the impoundment is about elevation 655 feet at the upstream face of the main embankment. There are three dikes, or "floating roads," across the impoundment. The dikes appear to have elevations of about 666 feet, 676 feet, 682 feet, from the most downstream to upstream, respectively. We were also provided with design drawings of the existing dewatering pipe, which has a tower riser with stop logs.

We further understand that the facility has been designed for a crest elevation of 711 feet and that an open channel spillway has been constructed with a bottom elevation of 706.25 feet for the proposed crest. We were provided with design drawings of that open channel spillway.

Task 1 - Hydrologic Analysis of Existing Facility

Analyses were performed to satisfy Kentucky Division of Water (KYDOW) requirements for a high hazard facility. The requirements specify a principal spillway pipe and emergency spillway must pass the 6-hour Probable Maximum Precipitation (PMP) below the crest elevation. The 6-hour PMP depth is 28.1 inches for Lawrence County, Kentucky. Furthermore, 80% of the storm storage must be decanted within 10 days of the peak stage.

After developing a rating curve for the existing dewatering structure, we performed a hydrologic analysis based on the conditions shown on the topographic mapping. The analysis was performed assuming an operating pool elevation of 655 feet (at the upstream face, higher above the floating roads), a crest elevation of 692 feet, and an inlet elevation of 660 feet for the dewatering structure. The rating curve and analysis results are provided in Appendix I. As shown, the dam is adequate to store/route the design storm with a maximum pool level of 676.2 feet during the storm, resulting in 15.8 feet of freeboard below the crest. Furthermore, 80% of the storm storage volume is decanted in less than 10 days.

A second analysis was performed to determine what the maximum operating pool level could be to store/route the design storm. That maximum operating pool level is elevation 682 feet, resulting in a pool level of 690.1 feet during the design storm and 1.9 feet of freeboard below the existing crest. These analysis results are also provided in Appendix I.

Task 2 - Development of Crest/Pool Rating Table

A rating curve was developed for the existing open channel spillway with bottom elevation 706.25 feet. The rating curve was developed using the USACOE's *HEC-RAS* River Analysis System. The rating curve is included in Appendix II.

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1*****************************************                              ***************************************
 *                                       *                              *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                              *   U.S. ARMY CORPS OF ENGINEERS      *
 *           SEPTEMBER 1990              *                              *   HYDROLOGIC ENGINEERING CENTER     *
 *            VERSION 4.0                *                              *         609 SECOND STREET           *
 *                                       *                              *      DAVIS, CALIFORNIA 95616        *
 *  RUN DATE  11/16/2006  TIME  09:21:04 *                              *          (916) 756-1104             *
 *                                       *                              *                                     *
*****************************************                              ***************************************


                         X     X  XXXXXXX   XXXXX          X
                         X     X  X        X     X        XX
                         X     X  X        X               X
                         XXXXXXX  XXXX     X        XXXXX  X
                         X     X  X        X               X
                         X     X  X        X     X         X
                         X -   X  XXXXXXX   XXXXX         XXX


          THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

          THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
          THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
          NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
          DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
          KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM


1                                                    HEC-1 INPUT                                              PAGE  1

        LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 *** FREE ***
          1           ID  ************************************************************************
          2           ID  *                                                                      *
          3           ID  *      Big Sandy Fly Ash Dam           File: bsexpmp.inp              *
          4           ID  *      Flood Routing for Existing 692 Crest Pool Elev 655             *
          5           ID  *      6 Hour PMP                                                     *
          6           ID  *      GA Project No. 06-365                                          *
          7           ID  *                                                                      *
          8           ID  *      Analyses by: Geo/Environmental Associates                      *
          9           ID  *                   Knoxville, TN                                     *
         10           ID  *                   November 2006                                     *
         11           ID  *                   SMA                                               *
         12           ID  ************************************************************************
         13           IT       6       0       0     300
         14           IO       3
         15           JR    FLOW     1.0
         16           VS   BASIN     IMP     IMP     IMP
         17           VV    2.11    2.11    6.11    7.11
         18           IN       6

         19           KK   BASIN
         20           KM   CALCULATED INFLOW USING 6 HOUR SCS DISTRIBUTION
         21           PB    28.1
         22           PC  .00000 0.00453 0.00906 0.01359 0.01812 0.02265 0.02729 0.03216 0.03725  0.0425
         23           PC  .04812 0.05396 0.06013 0.06664 0.07349 0.08068 0.08832 0.09653 0.10531  0.1146
         24           PC  .12456 0.13543 0.14766 0.16125 0.17619 0.19250 0.22624 0.29350 0.40168  0.5957
         25           PC  .73036 0.75720 0.78081 0.80119 0.81834 0.83227 0.84422 0.85543 0.86590  0.8756
         26           PC  .88464 0.89308 0.90112 0.90876 0.91601 0.92286 0.92937 0.93560 0.94154  0.9472
         27           PC  .95258 0.95778 0.96286 0.96786 0.97274 0.97754 0.98222 0.98682 0.99131  0.9957
         28           PC  1.0000 1.00000 1.00000 1.00000 1.00000
         29           BA   0.877
         30           LU       0    0.05       4
         31           UD       0

         32           KK     IMP
         33           KM   ROUTE COMPUTED HYDROGRAPH THROUGH IMPOUNDMENT
         34           RS       1    ELEV     655
         35           SA      21      24      35      82     153     158     160
         36           SQ       0       0     100     123     123     123     123
         37           SE     655     660     665     670     680     690     692
         38           ZZ
1*****************************************                              ***************************************
 *                                       *                              *                                     *
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Department for Natural Resources and Environmental Protection
Division of Water
Engineering Memorandum No. 5

SECTION C - HYDROLOGIC CRITERIA

II. PRINCIPAL SPILLWAY

The retarding storage and associated principal spillway discharge will be such that the emergency spillway will not operate more frequently than indicated in Table F-I, Section B, Emergency Spillways. The inflow hydrograph or the minimum runoff volume for developing the balance between principal spillway capacity and retarding storage will be determined by procedures in Chapter 21, Section 4, SCS National Engineering Handbook. In areas where streamflow records can be regionalized and transposed to ungaged watersheds (based on the volume-duration-probability analyses), the Division of Water will authorize the use of these data for developing the principal spillway capacity and retarding storage. When other streamflow data are used, sufficient documentation must be prepared to show how these values were determined.

In the determination of the retarding storage and the principal spillway capacity, it is assumed that the initial reservoir stage is at the crest of the principal spillway.

III. EMERGENCY SPILLWAY

The emergency spillway hydrograph will be routed through the reservoir starting with a water surface at the elevation of the principal spillway inlet or at the water surface elevation after 10 days of drawdown, whichever is higher. The 10-day drawdown will be computed from the maximum water surface elevation which would be attained during the passage of the minimum principal spillway design runoff for that class of structure.

IV. FREEBOARD

The freeboard hydrograph for class (A) and (B) structures will be routed through the reservoir starting at the same water surface elevation as for the emergency spillway hydrograph. The routing of the freeboard hydrograph for class (C) structures may be started at the crest of the principal spillway.

V. MINIMUM HYDROLOGIC CRITERIA

Minimum hydrologic criteria are established for the development of each hydrograph as follows:

Emergency Spillway Hydrograph

Class (A) $P_A = P_{100}$
Class (B) $P_B = P_{100} + 0.12 x (PMP - P_{100})$
Class (C) $P_C = P_{100} + 0.26 x (PMP - P_{100})$

Freeboard Hydrograph

Class (A) $P_A = P_{100} + 0.12 x (PMP - P_{100})$
Class (B) $P_B = P_{100} + 0.40 x (PMP - P_{100})$
Class (C) $P_C = PMP$

in which P denotes 6-hour design rainfall, P_{100} refers to 6-hour, 100-year precipitation, and PMP represents 6-hour Probable Maximum Precipitation.

The above values may be obtained from the "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years", Technical Paper No. 40, Weather Bureau, U. S. Department of Commerce, Washington, D. C., and "Two To Ten-Day Precipitation For Return Periods of 2 To 100 Years In Contiguous United States", Technical Paper No. 49, Weather Bureau, U. S. Department of Commerce, Washington, D. C. These values may also be found in Division of Water, Kentucky Department for Natural Resources and Environmental Protection, Engineering Memorandum No. 2, "Rainfall Frequency Values for Kentucky."

When hydrographs are required for drainage areas with times of concentration in excess of 6 hours, the above must be modified to reflect the appropriate storm period.

The establishment of the above criteria does not eliminate the need for sound engineering judgment but only establishes the lowest limit of design considered acceptable.

It is the responsibility of the design engineer to classify the structure and to determine if the design requirements are in excess of the minimum.

RAINFALL FREQUENCY VALUES FOR KENTUCKY

Engineering Memorandum No. 2

April 30, 1971; Revised - June 1, 1979

COMMONWEALTH OF KENTUCKY

DEPARTMENT FOR NATURAL RESOURCES AND ENVIRONMENTAL PROTECTION

BUREAU OF NATURAL RESOURCES

DIVISION OF WATER RESOURCES

KENTUCKY

DIVISION OF WATER RESOURCES
DEPARTMENT FOR NATURAL RESOURCES AND ENVIRONMENTAL PROTECTION
ENGINEERING MEMORANDUM NO.2 (4-30-71), REVISED (6-1-79)

6 HOUR RAINFALL (INCHES)

FREQUENCY (YEARS)

COUNTY	1	2	5	10	25	50	100	PMP
GRANT	1.9	2.2	2.7	3.1	3.5	4.0	4.3	27.7
GRAVES	2.2	2.6	3.3	3.7	4.1	4.7	5.0	28.9
GRAYSON	2.1	2.4	3.0	3.4	3.9	4.4	4.7	28.5
GREEN	2.0	2.3	2.9	3.4	3.8	4.3	4.6	28.6
GREENUP	1.8	2.1	2.5	3.0	3.5	3.8	4.1	27.7
HANCOCK	2.1	2.4	3.0	3.4	3.9	4.3	4.7	28.2
HARDIN	2.0	2.3	2.9	3.4	3.8	4.3	4.6	28.3
HARLAN	1.9	2.3	2.8	3.3	3.8	4.3	4.7	28.8
HARRISON	1.9	2.2	2.7	3.1	3.6	4.0	4.3	27.9
HART	2.0	2.3	2.9	3.4	3.9	4.4	4.6	28.6
HENDERSON	2.1	2.5	3.0	3.5	4.0	4.4	4.8	28.3
HENRY	1.9	2.2	2.8	3.2	3.6	4.1	4.4	27.9
HICKMAN	2.3	2.6	3.3	3.7	4.2	4.7	5.1	29.0
HOPKINS	2.1	2.5	3.1	3.5	4.0	4.5	4.8	28.6
JACKSON	1.9	2.3	2.8	3.2	3.8	4.2	4.5	28.5
JEFFERSON	2.0	2.3	2.8	3.3	3.7	4.2	4.5	28.0
JESSAMINE	1.9	2.2	2.8	3.2	3.7	4.1	4.4	28.2
JOHNSON	1.8	2.2	2.6	3.1	3.7	3.9	4.4	28.2
KENTON	1.8	2.1	2.7	3.1	3.5	3.9	4.2	27.5
KNOTT	1.9	2.2	2.7	3.2	3.8	4.0	4.5	28.5
KNOX	1.9	2.3	2.9	3.3	3.8	4.4	4.7	28.8
LARUE	2.0	2.3	2.9	3.4	3.8	4.3	4.6	28.4
LAUREL	1.9	2.3	2.8	3.3	3.8	4.3	4.6	28.7
LAWRENCE	1.8	2.1	2.6	3.0	3.6	3.9	4.3	28.1
LEE	1.9	2.2	2.7	3.2	3.7	4.1	4.4	28.4
LESLIE	1.9	2.3	2.8	3.3	3.8	4.2	4.6	28.7
LETCHER	1.9	2.3	2.8	3.2	3.8	4.1	4.6	28.6
LEWIS	1.8	2.1	2.6	3.0	3.5	3.9	4.2	27.8
LINCOLN	2.0	2.3	2.8	3.3	3.8	4.3	4.5	28.5
LIVINGTON	2.2	2.5	3.2	3.6	4.1	4.6	5.0	28.6
LOGAN	2.1	2.4	3.1	3.5	4.0	4.5	4.8	28.9
LYON	2.2	2.5	3.2	3.6	4.1	4.6	4.9	28.8
MCCRACKEN	2.2	2.6	3.2	3.7	4.1	4.6	5.0	28.7
MCCREARY	2.0	2.4	2.9	3.4	3.9	4.4	4.7	28.9
MCCLEAN	2.1	2.4	3.0	3.5	3.9	4.4	4.8	28.5
MADISON	1.9	2.2	2.8	3.2	3.7	4.2	4.4	28.3
MAGOFFIN	1.8	2.2	2.7	3.1	3.7	4.0	4.4	28.3
MARION	2.0	2.3	2.9	3.3	3.8	4.3	4.5	28.4
MARSHALL	2.2	2.6	3.2	3.7	4.1	4.6	5.0	28.8
MARTIN	1.8	2.1	2.6	3.0	3.7	3.9	4.4	28.2