Puerto Rico Transportation Technology Transfer Center
Department of Civil Engineering and Surveying
University of Puerto Rico at Mayagüez
Call Box 9000 * Mayagüez, PR 00681
Tel. 787-834-6385 * Fax: 787-265-5695 * www.prltap.org

29th Anniversary of Excellence in the Training of Transportation Officials at Municipal, State and Federal Level in Puerto Rico and Virgin Islands

URBAN DRAINAGE DESIGN

Part 1

Instructor

Dr. Walter F. Silva-Araya
Professor
Department of Civil Engineering and Surveying
University of Puerto Rico – Mayagüez Campus

February 12 & 13, 2015
St. Thomas, US Virgin Islands

INDEX

Module 1

Hydrology: Rainfall and Design Storms

PRECIPITATION & HYDROLOGIC ABSTRACTIONS

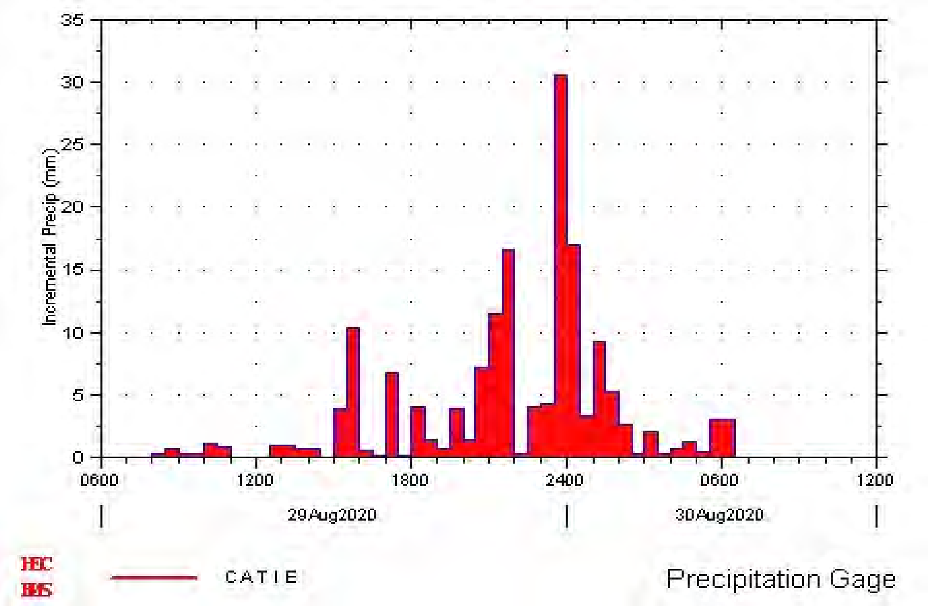
Walter F. Silva, Ph.D.

UPR Mayaguez

Characteristics of precipitation

- Precipitation changes in space and time according to wind circulation and local factors.
- It could take the form of rainfall, snow or hail.
- Rainfall is represented in isohyetal maps. These are maps with contour lines of equal precipitation.
- The lines are usually interpolated from point values measured at different stations.

Characteristics of precipitation



Hyetograph

- A pluviograph measures the rainfall as a function of time
- A hyetograph is a column chart presenting the distribution of precipitation as a function of time
- Rainfall Depth is the volume of rainfall divided by the surface area where it occurs

◆Hyetographs present precipitation amounts as rainfall depth (inch or mm) or as rainfall intensity (in/hr, mm/hr)

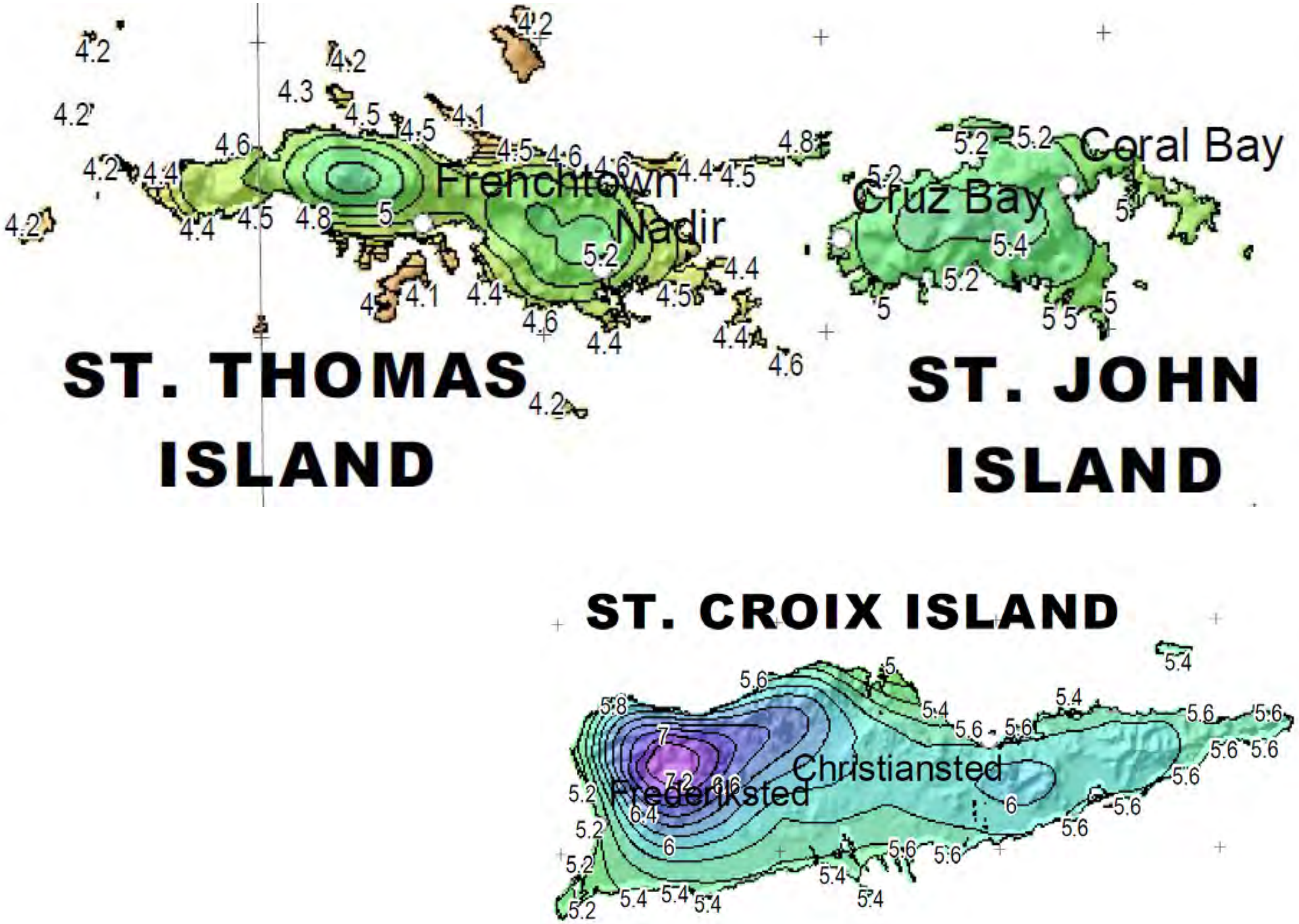
Characteristics of precipitation

- The rainfall event could be of two types:
    - Real (historic) storms
    - Design storms
- Real storm events are used for analysis of rainfall and detection of statistical trends in a region.
- Design storms are hypothetical storms with an associated probability of occurrence and return period.
- Design storms could be derived by frequency analysis from historic rainfalls, if a long period of data is available.

Methods for calculation of Areal Precipitation

- ***Isohyetal method*:**
  - Isohyets are contour lines of equal precipitation depth
  - Are constructed by interpolation between rainfall stations.
  - The area between two contour lines is the weighting factor for estimating the average precipitation
  - The rainfall assigned to each area is the average between the rainfall depth of two consecutive lines
  - Requires a dense stations network, but, orographic and local factors could be taken into consideration.

Isohyets Map



SCALE 1:600,000 (when printed/viewed at ANSI C size)

0 3 6 9 12 Miles

0 2.5 5 10 15 20 Kilometers

Isopluvials of 6 hour precipitation (inches)
with Average Recurrence Interval of 10 years

See NOAA Atlas 14 documentation for factors to convert to Annual
Exceedance Probabilities for all estimates below 25 years

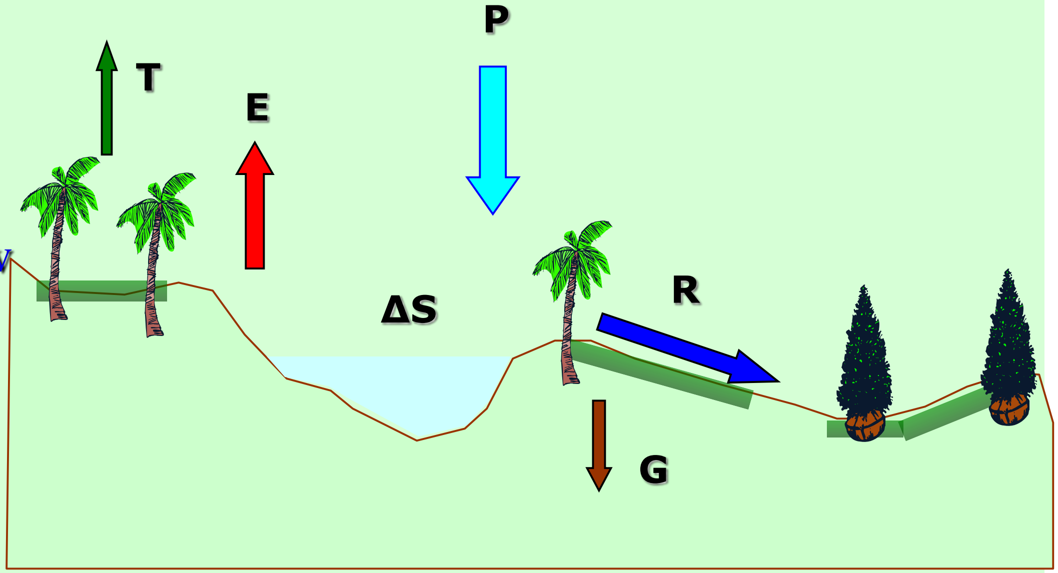
Water Budget

- The basic equation for solution is any hydrologic balance is:

$$P - R - G - E - T = \Delta S$$

Where:

- P =precipitation
- ΔS = storage
- G = groundwater flow (infiltration)
- R = runoff
- E = evaporation
- T = transpiration



Evaporation and Evapotranspiration

- NWS Class A pan method
  - Is a 4 ft diameter, 10 in deep pan
  - Made of unpainted galvanized iron
  - Measures evaporation directly
  - Uses a coefficient K (between 0.6 and 0.8 with an average of 0.7) as correction factor
  - $E_{real} = KE_{pan}$
  - There are refinements of this method

NWS Class A pan

Evaporation

The following data are mean daily pan evaporation measurements (in/day). Assuming a pan coefficient of 0.68 , estimate the daily lake evaporation rates (acre-ft and in) for a 27-acre lake

$$E = c_{pan} E_p$$

$$C_{pan} = \text{pan coefficient}$$

$$E_p = \text{pan evaporation (in)}$$

- Conversion Factor from inches to acre-ft: E (in) x Area (acres)/12
- Example for Day 1: E = 0.68 x 0.22 = 0.15 in = 0.15 x 27/12 = 0.337 ac-ft

Day	1	2	3	4	5	6	7
Pan E (in)	0.22	0.26	0.25	0.28	0.26	0.21	0.22
ELake (in)	0.150	0.177	0.170	0.190	0.177	0.143	0.150
E Lake (ac-ft)	**0.337**	**0.398**	**0.383**	**0.428**	**0.398**	**0.321**	**0.337**

Definitions

Losses:

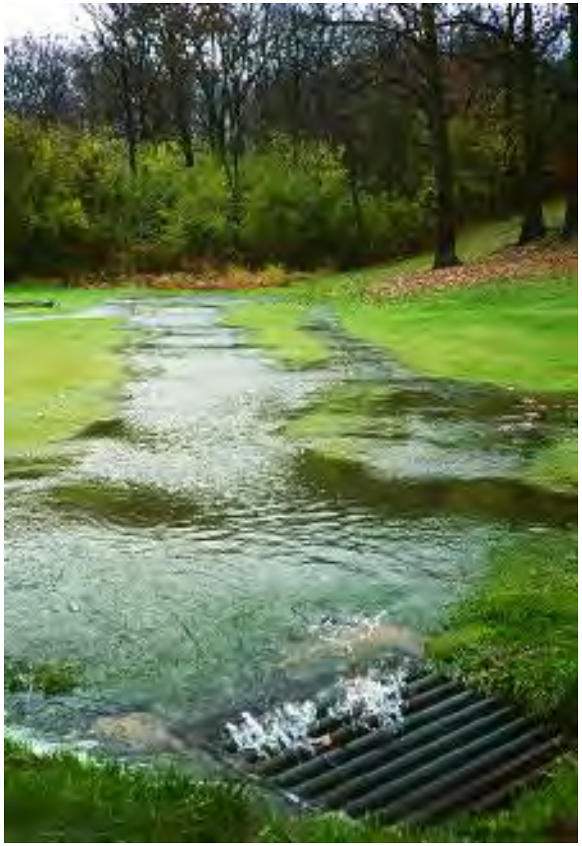
- **Interception storage**: intercepted by vegetation
- **Depression storage:** water stored in small surface depressions
- **Infiltration or soil storage**: water that infiltrates into the soil

Definitions

- Interception storage and depression storage are depleted during the early part of the storm, therefore, are part of the initial abstractions
- The primary component of losses is infiltration of rainfall into subsurface storage
- The volume of subsurface storage is greatest at the start of rainfall and decreases over the duration of the storm

Definitions

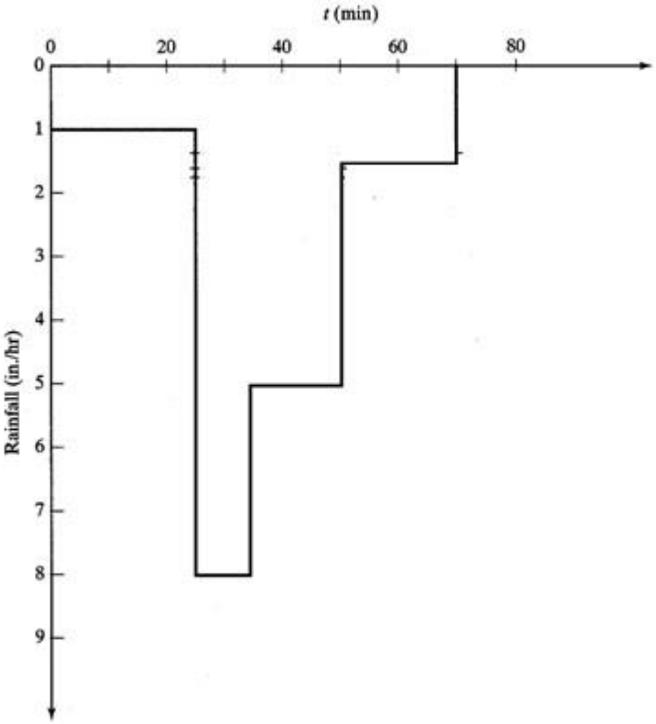
- Rainfall Excess: volume of rain that moves overland and becomes direct runoff
- Losses are the difference between the total rainfall and the direct runoff
- It is common to express rainfall and losses in inches or mm



A rainfall hyetograph for a 70 min storm with a total depth of 3.5 in is given. The depth of direct runoff is 1.90 in.

How much is the rainfall excess

By definition the rainfall excess is the same as the direct runoff: $V_d = 1.9$ in



How much is the depth of losses

The losses are the difference between the total rainfall depth and the direct runoff. Depth of losses $V_L = 3.5 - 1.9 = 1.6$ in

Precipitation

Given the following depths of rainfall for a 50 min storm, how much is the maximum 30 min intensity

Storm Time (min)	10	20	30	40	50
Rainfall Depth (in)	0.05	0.2	0.3	0.25	0.1

- Convert depth to intensity: Divide by the interval and include conversion factors

- Examples:
- $\Delta t = 10$ min, $P = 0.05$ in at first 10 min.
- $I_{10min} = 0.05 / (10/60) = 0.3$ in/hr

- $\Delta t = 30$ min, $P = 0.05+0.2+0.3 = 0.55$ in
- $I_{30min} = 0.55 / (30/60) = 1.5$ in/hr

Time (min)	Rainfall (in)	10 min Intensity (in/hr)	inches in 30 min	30 min Intensities (in/hr)	
10	0.05	**0.3**			
20	0.2	0.8			
30	0.3	1.2	**0.55**	1.1	
40	0.25	0.75	**0.75**	**1.5**	**MAX I**
50	0.1	0.24	0.65	1.3	

- Add depths in groups of 30 min intervals (column 4) and divide by 0.5 hr.
- The maximum 30 min intensity occurs during the middle three ordinates
- The total rainfall during this period was $0.2 + 0.3 + 0.25 = 0.75$ in
- The intensity is: $i_{30} = 0.75/(30/60) = 1.5$ in/hr

Precipitation

Given the following depths of rainfall for a 8 hr storm, what is the maximum 4 hr intensity

Storm Time (min)	2	4	6	8
Rainfall Depth (in)	0.7	1.5	0.9	0.5

- Convert depth to intensity: Divide by the interval and include conversion factors

- Examples:
- $\Delta t = 2$ hr, $P = 0.7$ in at first 2 hrs.
- $I_{2hrs} = 0.7 / 2 = 0.35$ in/hr

- $\Delta t = 4$hr, $P = 0.7+1.5 = 2.2$ in
- $I_{4hrs} = 2.2 / 4 = 0.55$ in/hr

Time (hr)	Rainfall (in)	2 hr Intensities in/hr	inches in 4 hr	4 hr Intensities (in/hr)	
2	0.7	0.35			
4	1.5	0.75	2.2	0.55	
6	0.9	0.45	**2.4**	**0.6**	**MAX I**
8	0.5	0.25	1.4	0.35	

- Add depths in groups of 4 hours intervals (column 4) and divide by 4 hr
- The maximum 4 hr intensity occurs during the middle two ordinates
- The total rainfall during this period was $1.5 + 0.9 = 2.4$ in
- The intensity is: $i_{4hr} = 2.4/4 = 0.6$ in/hr

Precipitation

Given the following depths of rainfall for a 45-min storm, how much is the total rainfall and the maximum intensity

Storm Time (min)	0-6	6-18	18-21	21-30	30-36	36-45
Rainfall Depth (in)	0.06	0.24	0.18	0.54	0.3	0.18

- To convert depth to intensity: Divide by the interval and include conversion factors

- Example for the interval from 6 to 18 min
- $\Delta t = 18\text{-}6 = 12\text{min} = 12/60 = 0.2\text{hr}$
- P = 0.24 in in 12 min.
- $I_{12min} = 0.24 / 0.2 = 1.2$ in/hr

- Example for the interval from 18 to 21 min
- $\Delta t = 3$ min $= 3/60 = 0.05$ hr
- P = 0.18 in
- $I_{3min} = 0.18 / (3/60) = 3.6$ in/hr

Storm Time (min)	0-6	6-18	18-21	21-30	30-36	36-45
Rainfall Depth (in)	0.06	0.24	0.18	0.54	0.3	0.18
DT (hr)	0.6	0.2	0.05	0.15	0.1	0.15
I (in/hr)	0.1	1.2	3.6	3.6	3	1.2

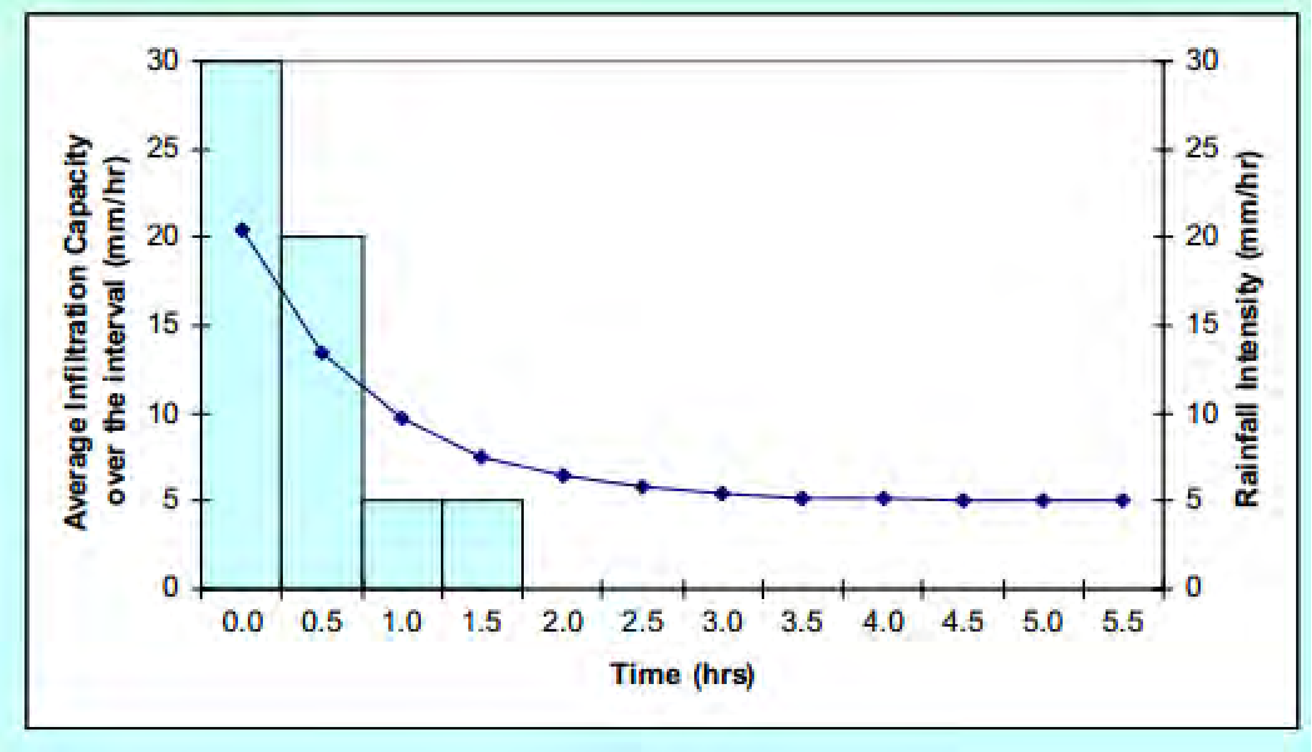
- The total rainfall during this period was 1.5 in
- The maximum intensity was 3.6 in/hr.

Separation of rainfall soil losses

- Reflect the ability of the watershed to retain water
- No rainfall: vegetation is dry, depressions are empty, upper layers of soil have low moisture: greatest potential for storage of rainwater

Infiltration capacity curve

- After certain time, the infiltration capacity approaches a constant value



Infiltration Methods

- There are several equations used to estimate infiltration.
- The Φ-index
- Horton
- Green-Ampt

Infiltration

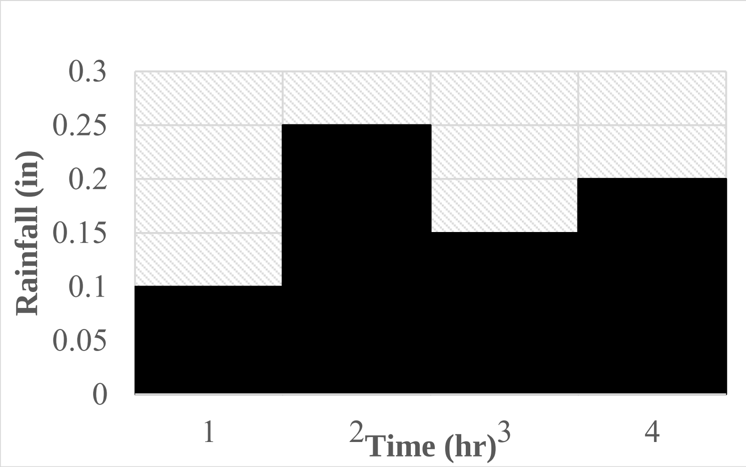
◆ The Φ-index method:

1. *Plot the overall precipitation rate versus time.*
2. *A horizontal line called the Φ-index is drawn on the plot, such that the volume of rainfall excess above this line is equal to the actual volume of observed runoff.*
3. *The index indicates the average infiltration rate for the storm event*

The Φ-index method

Example problem:

A hyetograph for a 4-hr storm is provided. The direct runoff is 0.4 in. Which is the Ph
Index in in/hr:



Estimate the Phi Index

$$\emptyset = \frac{Vp - Vd}{D} = \frac{(0.7 - 0.4)in}{4\ hr} = 0.075\ in/hr$$

Compute the rainfall excess substracting the loss (0.075 in.hr) from each ordinate of the hyetograph

Rainfall excess = *(0.1-0.075)+(0.25-0.075)+(0.15-0.075) + (0.2-0.075) = 0.4 in*

This value is the same as the direct runoff; therefore, the Φ is appropriate for this storm event.

Infiltration

- The Horton infiltration equation is:

$$f = f_c + (f_0 - f_c)e^{-kt}$$

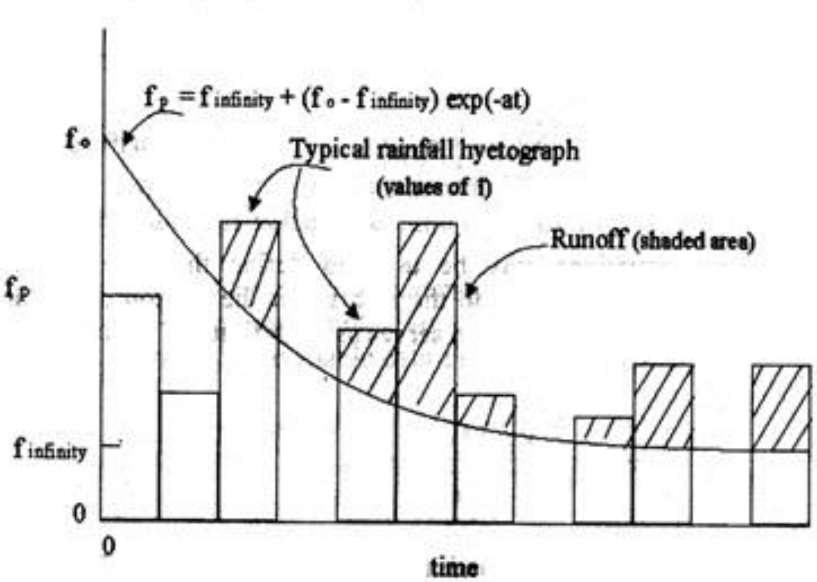
f =infiltration rate (in/hr)

f_o= initial infiltration rate (in/hr)

f_c= final infiltration rate (in/hr)

t = time (hr)

k = empirical constant (1/hr)



Horton Infiltration Curve and Typical Hyetograph.
For the case illustrated, runoff would be intermittent.

Infiltration

The Horton infiltration equation can be integrated to obtain the volume of infiltration as:

$$F = f_c t + \frac{f_0 - f_c}{K}\left(1 - e^{-kt}\right)$$

F = *total* infiltration volume during t hrs (in)

f_o = initial infiltration rate (in/hr)

f_c = final infiltration rate (in/hr)

t = time (hr)

k = empirical constant (1/hr)

The infiltration characteristics of a small watershed has the following parameters for the Horton Infiltration model: $f_0 = 0.6$ in/hr, $f_c = 0.2$ in/hr, and K = 0.6 1/hr. What is the total depth of infiltration after a period of 4 hours?

Solve using the integrated Horton's equation:

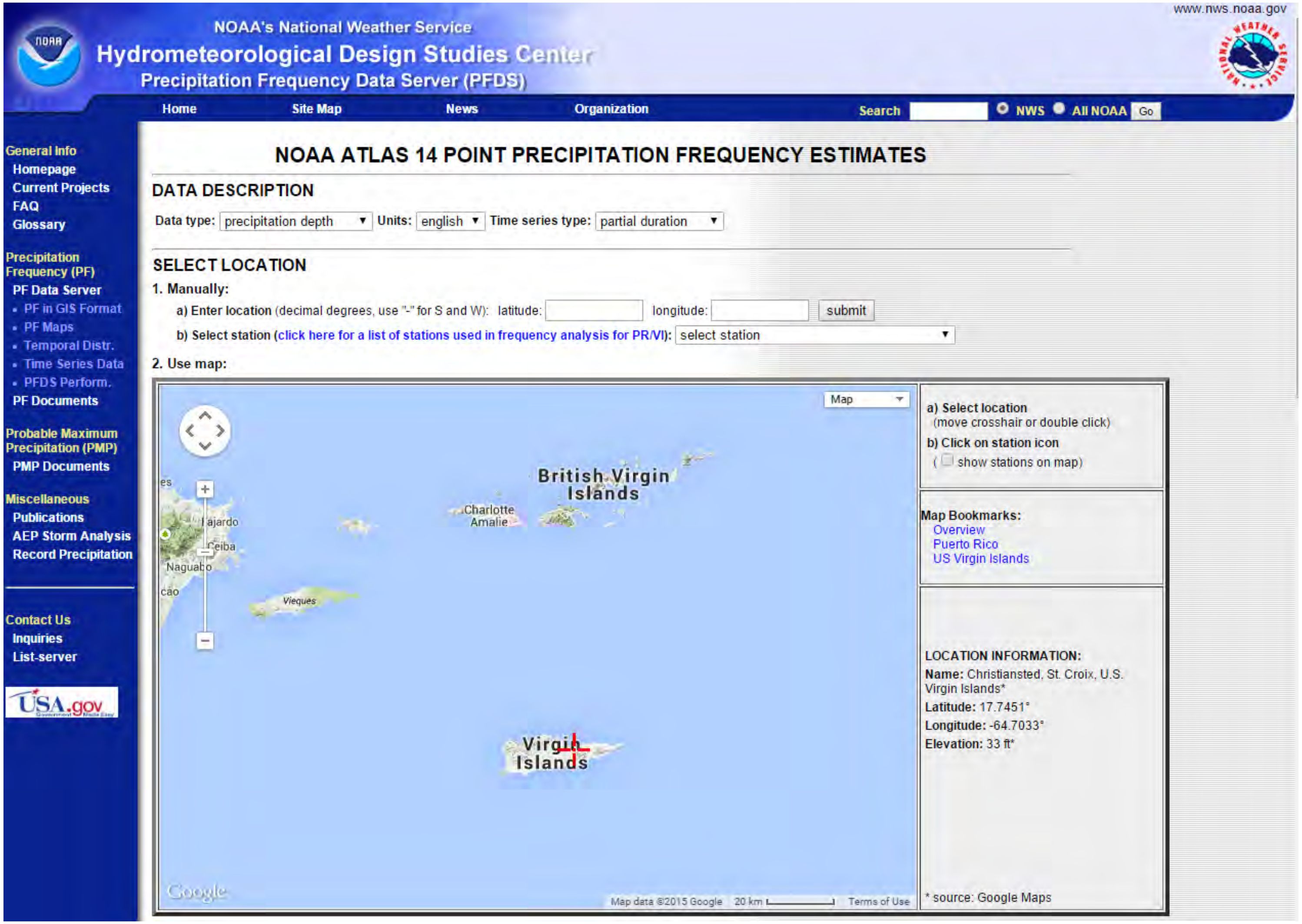
$$\mathrm{F} = \mathrm{f_c t} + \frac{\mathrm{f_0 - f_c}}{\mathrm{K}}\left(1 - \mathrm{e}^{-\mathrm{kt}}\right) = 0.2x4 + \left(\frac{0.6-0.2}{0.6}\right)x(1 - e^{-0.6x4}) = 1.406\ in$$

Intensity-duration-frequency

- Rainfall Intensity: Amount of precipitation per unit time (usually hours)
- The instantaneous intensity changes
- The annual probability of occurrence of a rainfall event is a design criterion (ex. 1% storm)
- The inverse of annual probabilty is called the return period or the recurrence interval (1% storm = 100 yrs storm)
- A 1% storm has a 1% chance of occurring in any particular year

Precipitation data for the U.S. Virgin Islands

- http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_pr.html



NOAA ATLAS 14 POINT PRECIPITATION FREQUENCY ESTIMATES

DATA DESCRIPTION

Data type: precipitation depth ▾ Units: english ▾ Time series type: partial duration ▾

SELECT LOCATION

1. Manually:
   a) Enter location (decimal degrees, use "-" for S and W): latitude: longitude: submit
   b) Select station (click here for a list of stations used in frequency analysis for PR/VI): select station ▾
2. Use map:

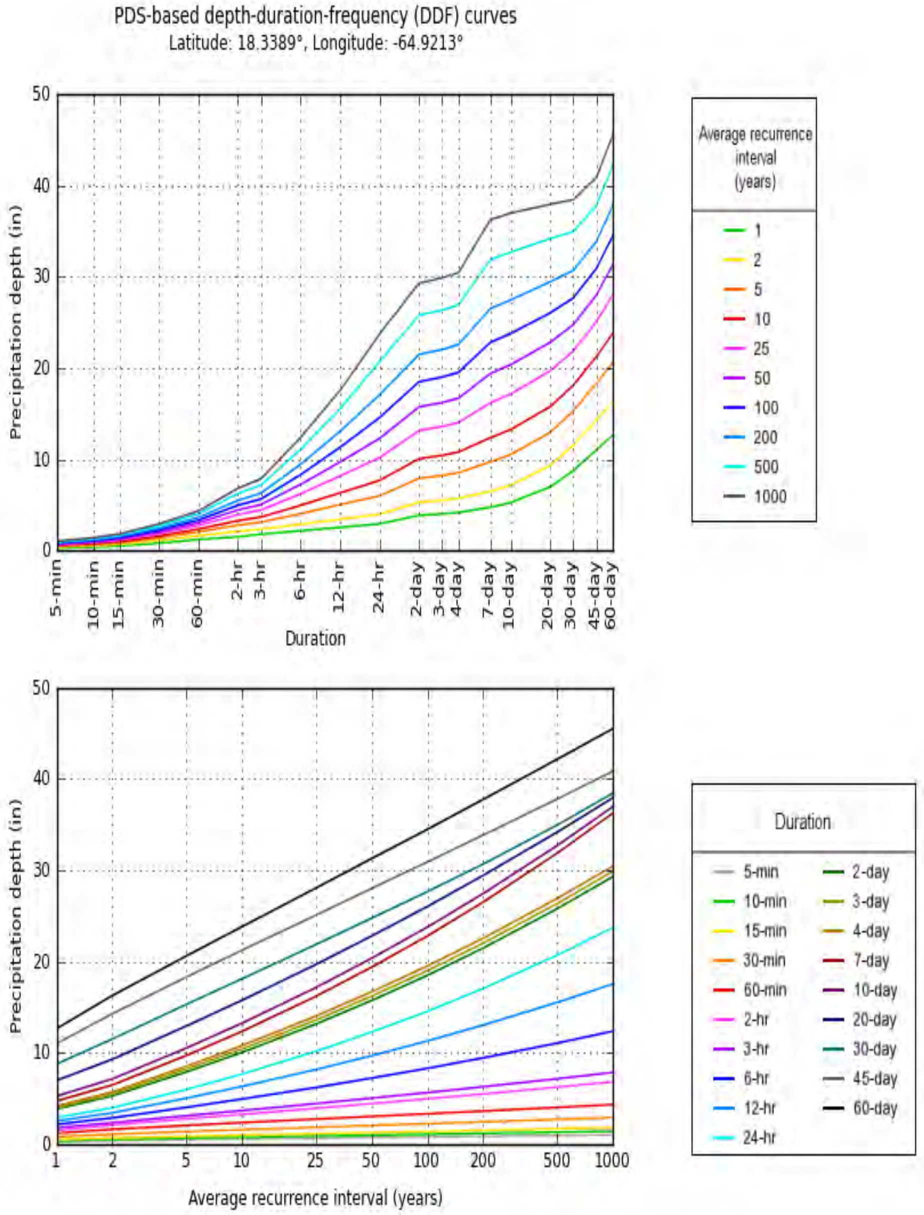
a) Select location
(move crosshair or double click)
b) Click on station icon
(☐ show stations on map)

Map Bookmarks:
Overview
Puerto Rico
US Virgin Islands

LOCATION INFORMATION:
Name: Christiansted, St. Croix, U.S. Virgin Islands*
Latitude: 17.7451°
Longitude: -64.7033°
Elevation: 33 ft*

* source: Google Maps

NOAA ATLAS 14 POINT PRECIPITATION FREQUENCY ESTIMATES


PDS-based depth-duration-frequency (DDF) curves
Latitude: 18.3389°, Longitude: -64.9213°

Intensity-duration-frequency

- The following mathematical forms for representing IDF curves are commonly used in hydrologic computations:

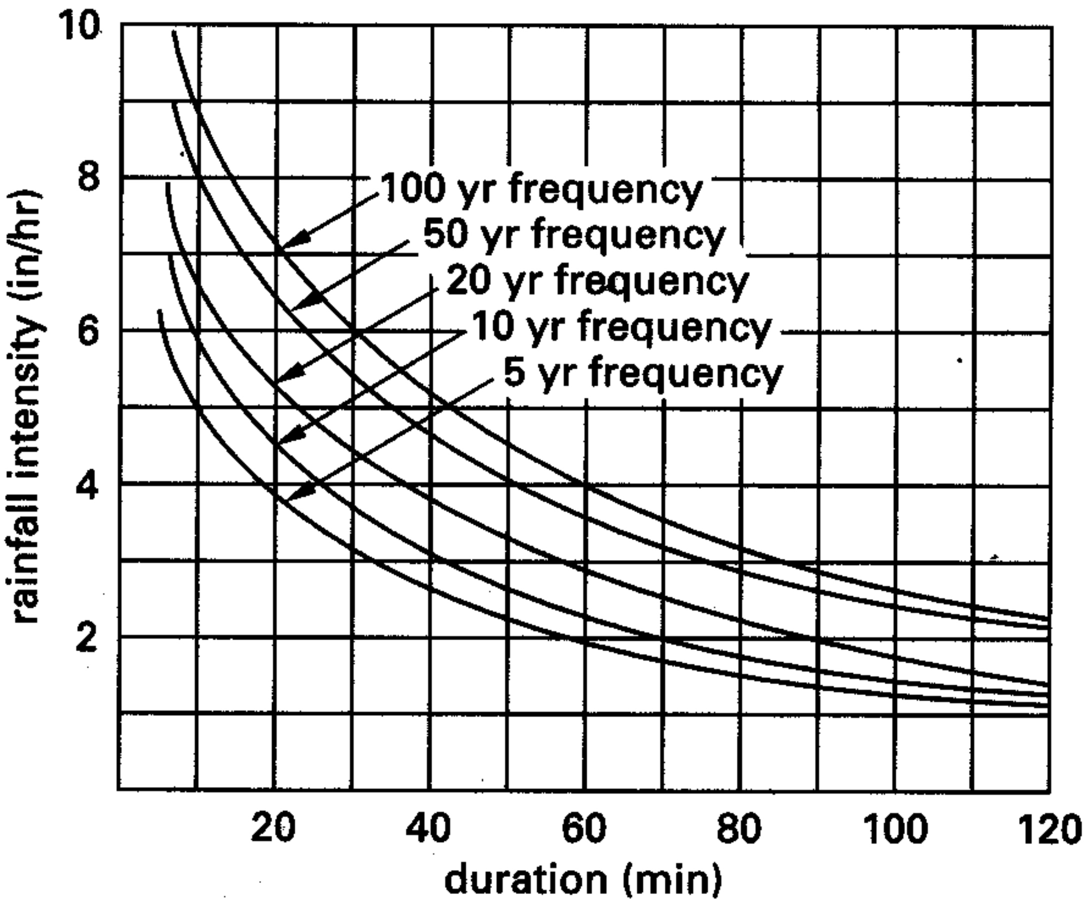
$$i = \frac{a}{D+b} \; for \;\; D \leq 2\, hr$$

$$i = cD^{d} \quad for \;\; D \geq 2\, hr$$

- a, b, c and d are constants, D is the storm duration
- Both equations can be linearized to obtain the constants

Intensity-duration-frequency

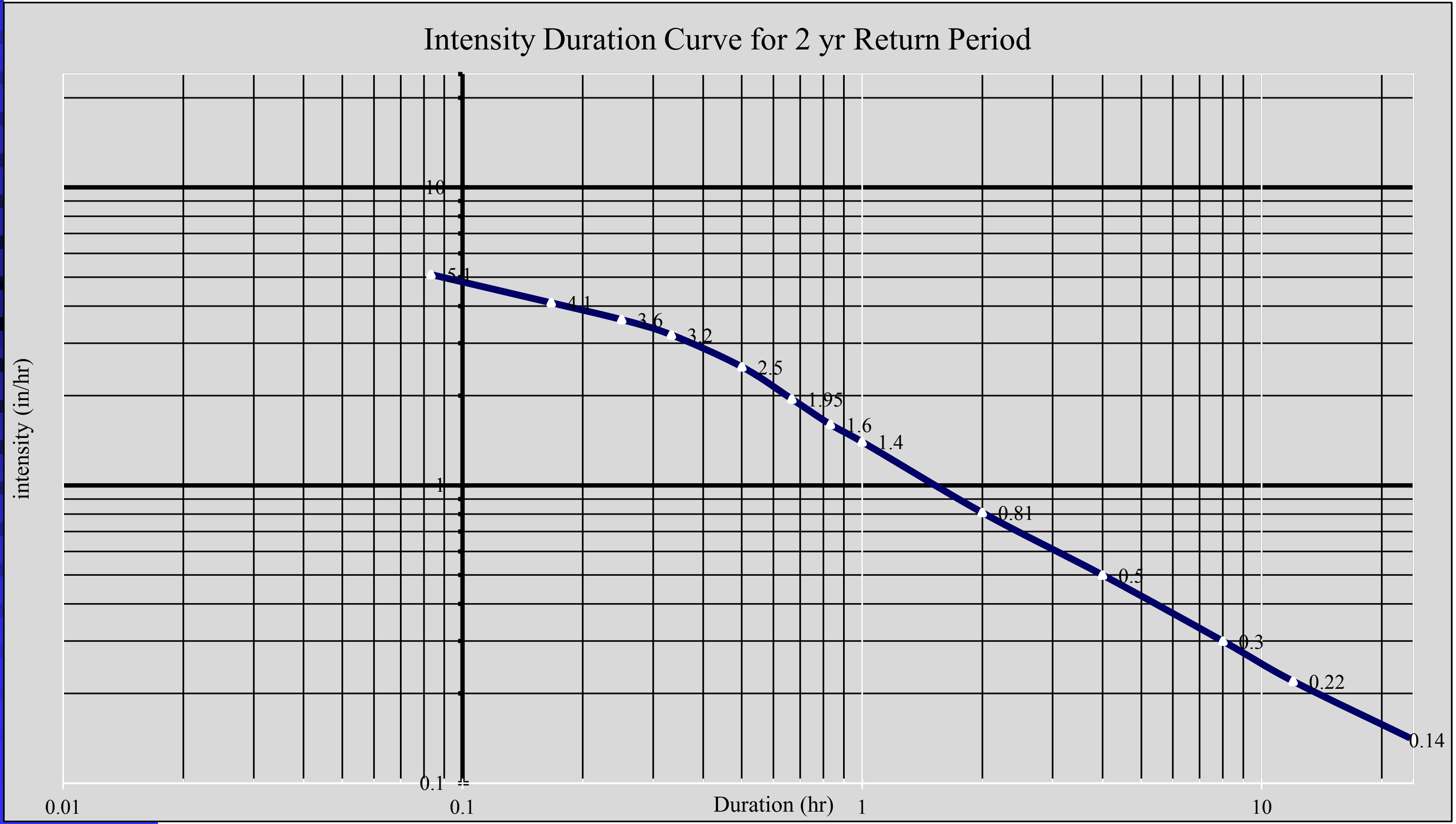
- Typical IDF curves:



Typical Intensity-Duration-Frequency Curves

Intensity-duration-frequency

You are provided a 2-yr Intensity-Duration Curve.



Intensity-duration-frequency

What are the coefficients *a* and b for the IDF curve approximation for a duration smaller than 2-hrs?

Ans. 1.80 and 0.28

What are the coefficients *a* and b for the IDF curve approximation for a duration greater than 2-hrs?

Ans. 1.37,-0.74

Variable transformations

$$i = \frac{a}{b + D}$$

$$\frac{1}{i} = \frac{b + D}{a}$$

$$y = f + gD$$

$$i = cD^d$$

$$\log i = \log c + d \log D$$

$$y = h + dx$$

Synthetic Rainfall Events

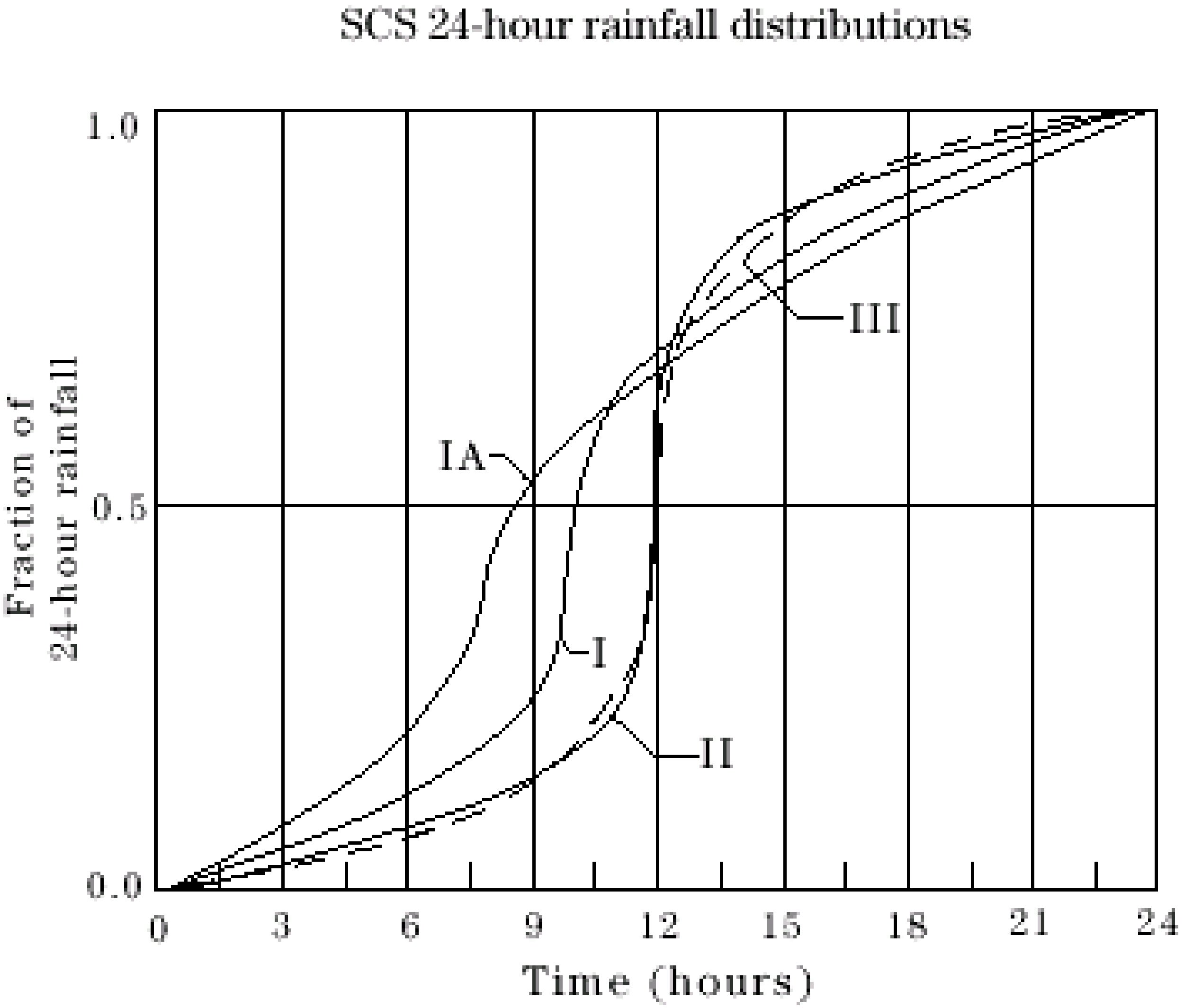
1. NRCS (SCS) 24 hour rainfall distributions
   - Developed by the USDA, NRCS
   - Four synthetic 24-hr rainfall distributions
   - Useful for all storm durations
   - Distributions apply according to geographic location
   - The critical parameter is the time of concentration.
     - Tc = Sum (Time of travel for segments)

$$T_t = L/V$$

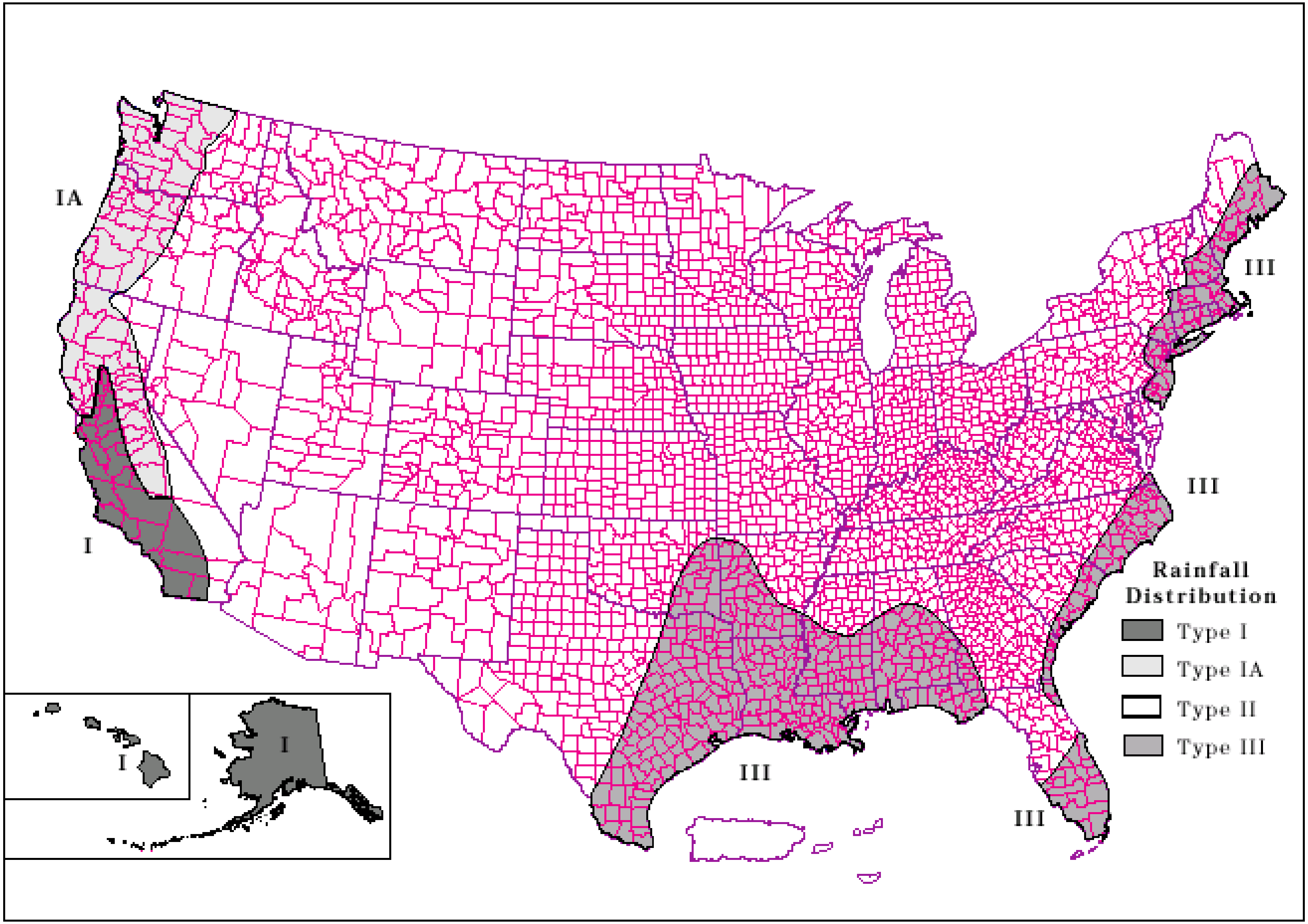
2. Rainfall distributions for any duration and return period can be obtained from NOAA Atlas 14 data

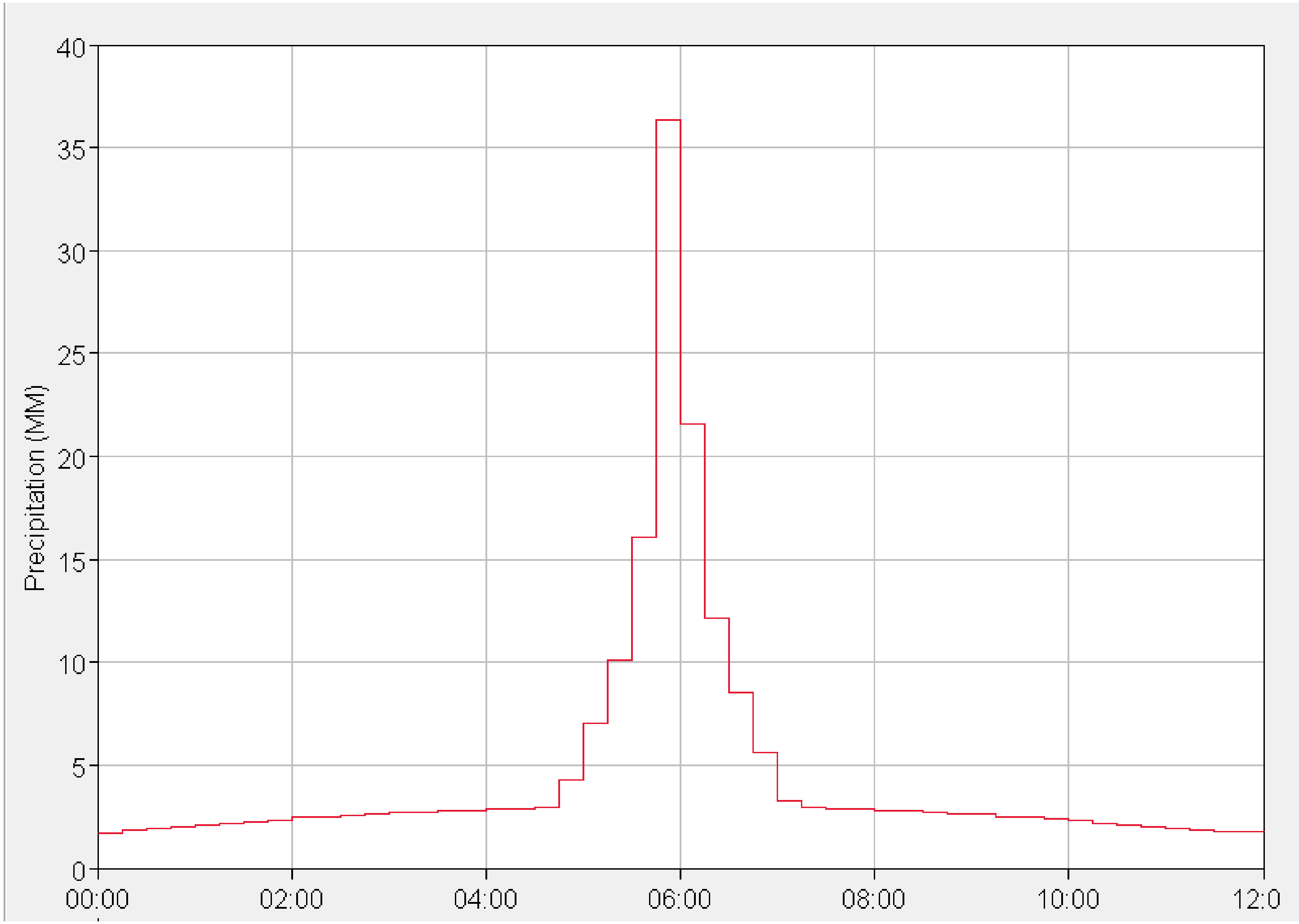
Synthetic Rainfall Events

NRCS (SCS) 24 hour rainfall distributions



Approximate geographic boundaries for NRCS (SCS) rainfall distributions



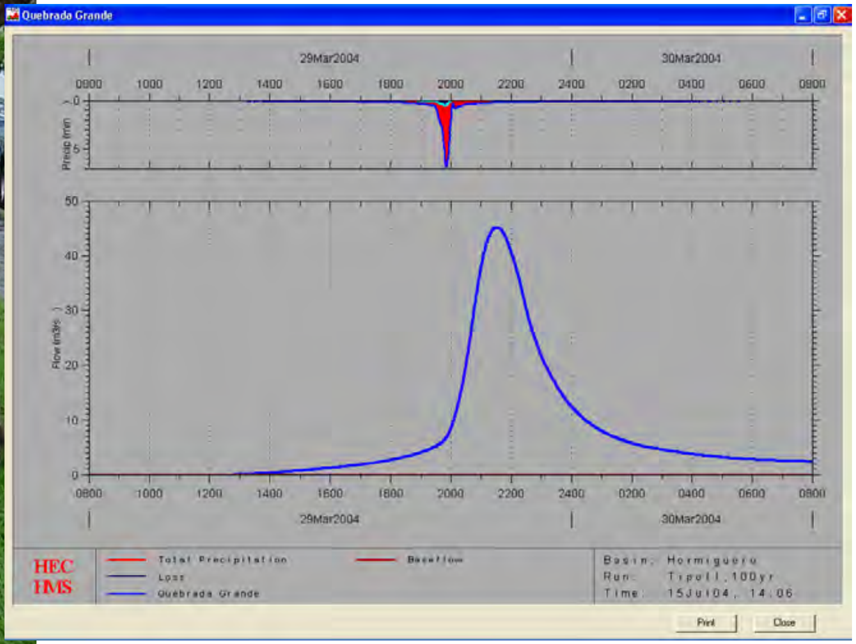


- Example: 25 yrs – 12 hours rainfall distribution in San German, PR, using data from NOAA Atlas 14

Module 2

Hydrology: Hydrograph and Peak Flow Estimation

Hydrographs and Peak Flow Estimation



Basin: Hormiguero
Run: Tipoll,100yr
Time: 15Jul04, 14:06

Dr. Walter F. Silva, UPRM

Runoff Volume

- Runoff= acumulation of rainfall excess due to precipitation
- Runoff volume = volume of water generated by runoff in a period of time. Sometimes is represented by the inches or mm of water over the watershed area.
- The rate of volume per unit time is the basis for the HYDROGRAPH. This is the time sequence of runoff, reported in units of discharge (m^3/s or ft^3/s).

Runoff Volume

- To estimate the runoff volume implies to consider the following variables
  - Precipitation
  - Infiltration
  - Evaporation
  - Transpiration
  - Interception
  - Depression storage
- All are complex processes

Runoff Estimation Methods

- Peak Discharge Methods
  - USGS Formulas: regression equations
  - Flood frequency analysis (moment estimation)
  - Index-flood
  - Rational Method
  - SCS (NRCS) Graphical Peak Discharge Method
  - Slope-Area Method for Discharge Estimation

PEAK DISCHARGE COMPUTATION

The Rational Method

- Is the most widely used method for estimation of peak discharge (Q_p) from runoff over small areas
- Assumes that a uniform rainfall (in space and time) occurs for a time long enough so the entire area contributes to the outflow
- Normally limited to areas below 1 sq mile (640 ac) (ASCE limits to 200 ac)

$$Q_p = CiA$$

- C = runoff coefficient (fraction of rain converted to runoff) (from table)
- *i* = Storm intensity corresponding to a duration equal to the time of concentration (in/hr) and for a design frequency
- A = watershed area (acres)

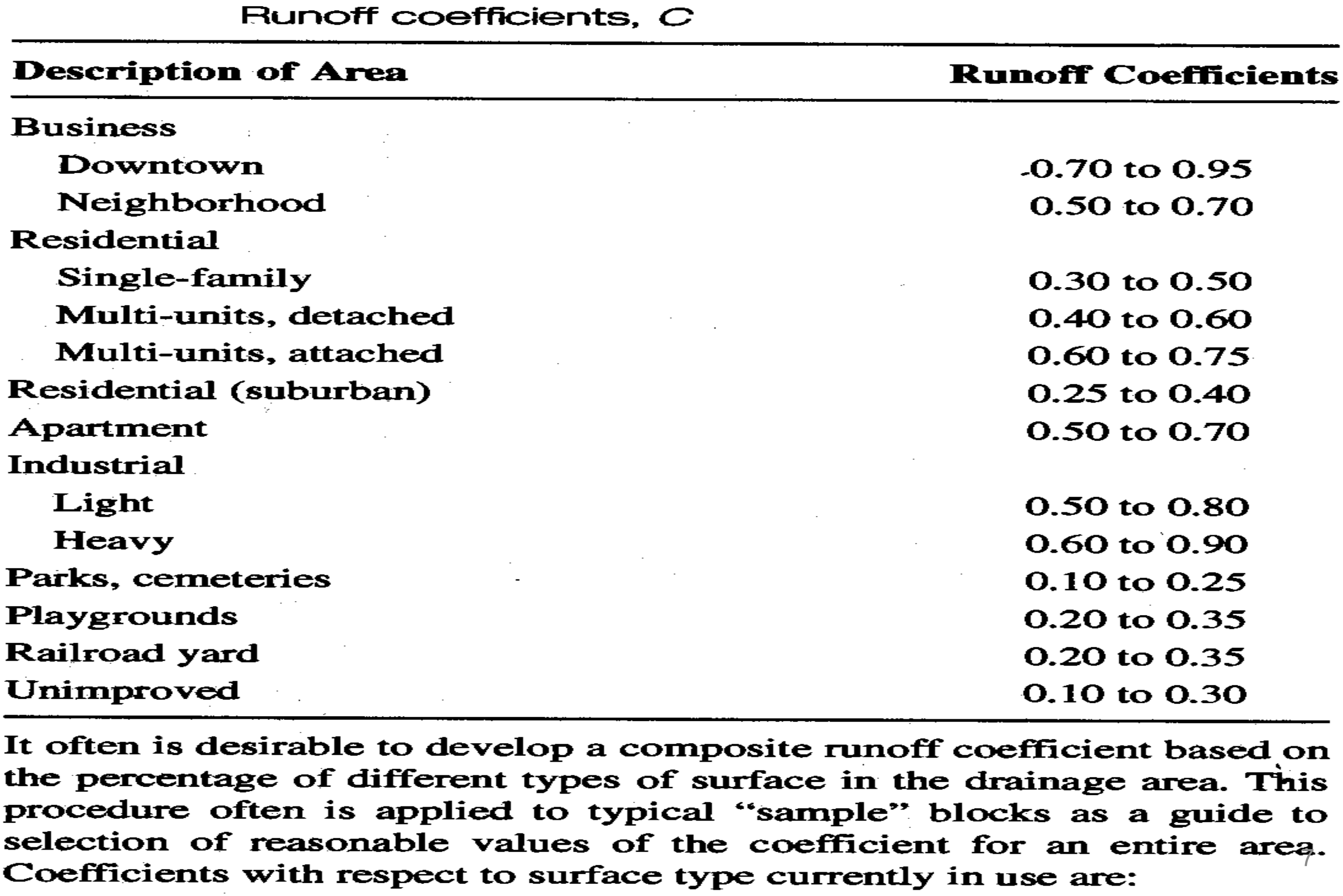
PEAK DISCHARGE COMPUTATION
The Rational Method

- Is the most widely used method for estimation of peak discharge (Q_p) from runoff over small areas
- Assumes that a uniform rainfall (in space and time) occurs for a time long enough so the entire area contributes to the outflow
- Normally limited to areas below 1 sq mile (640 acres) (ASCE limits to 200 ac)

$$Q_p = CiA$$

- C = runoff coefficient (fraction of rain converted to runoff) (from table)
- *i* = Storm intensity corresponding to a duration equal to the time of concentration (in/hr) and for a design frequency
- A = watershed area (acres)

Runoff Coefficients for Rational Formula

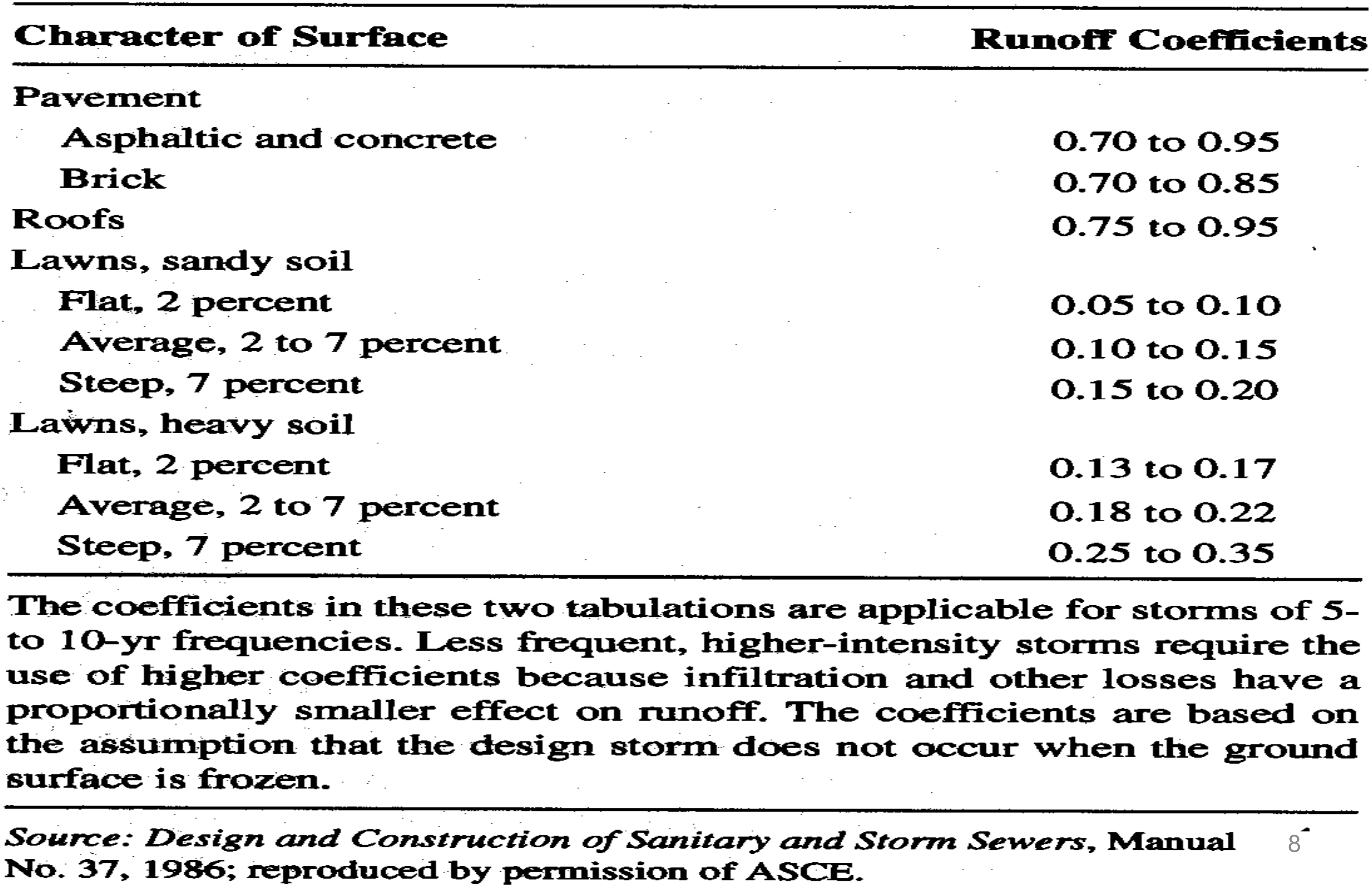


Runoff coefficients, C

Description of Area	**Runoff Coefficients**
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

It often is desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. This procedure often is applied to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area. Coefficients with respect to surface type currently in use are:

Runoff Coefficients for Rational Formula



Character of Surface	**Runoff Coefficients**
Pavement	
Asphaltic and concrete	**0.70 to 0.95**
Brick	**0.70 to 0.85**
Roofs	**0.75 to 0.95**
Lawns, sandy soil	
Flat, 2 percent	**0.05 to 0.10**
Average, 2 to 7 percent	**0.10 to 0.15**
Steep, 7 percent	**0.15 to 0.20**
Lawns, heavy soil	
Flat, 2 percent	**0.13 to 0.17**
Average, 2 to 7 percent	**0.18 to 0.22**
Steep, 7 percent	**0.25 to 0.35**

The coefficients in these two tabulations are applicable for storms of 5- to 10-yr frequencies. Less frequent, higher-intensity storms require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

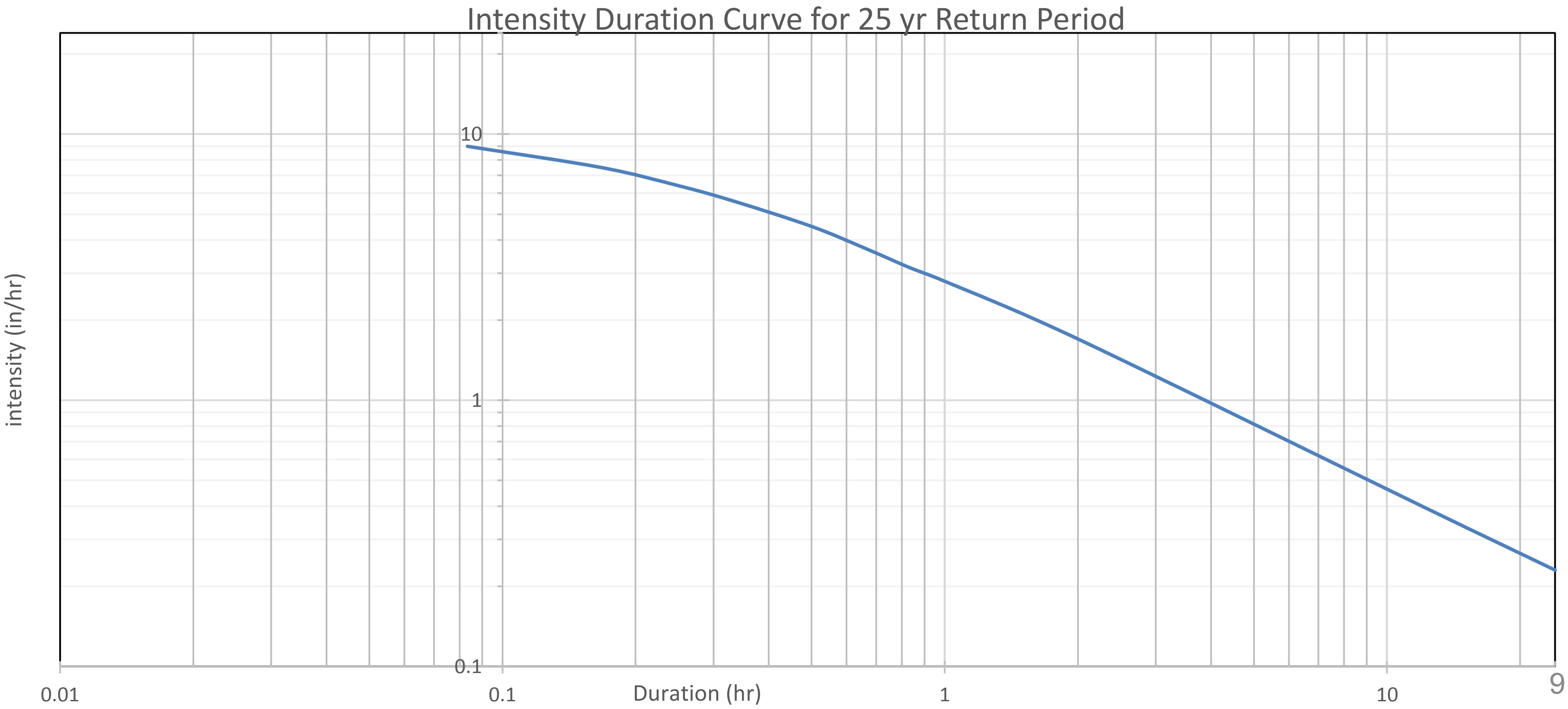
***Source: Design and Construction of Sanitary and Storm Sewers*, Manual No. 37, 1986; reproduced by permission of ASCE.**

PEAK DISCHARGE COMPUTATION

The Rational Method

It is required to design a pipe for 25-yr return period in a 2.4 acre parking lot in Mayaguez. The I-D curve for 25-yr is provided. The time of concentration is 6 min and the slope is 1.5%. Use the highest recommended runoff coefficient. The peak discharge for this design is most closely:

a) 14 b) 1.6 c) 20 d) 9



PEAK DISCHARGE COMPUTATION

The Rational Method: Non-homogeneous areas

- Where a watershed is non-homogeneous, a weighted runoff coefficient should be computed. The weighting factor is based on the area of each land use:

$$C_w = \frac{\sum_{j=1}^{n} C_j A_j}{\sum_{j=i}^{n} A_j}$$

  - C_w = weighted runoff coefficient
  - C_j = runoff coefficient for area *j*
  - A_j = Area for land cover *j*
  - *n*= number of land covers within the watershed
- The equation for the discharge becomes:

$$q_p = i \sum_{j=i}^{n} C_j A_j$$

- *i* = Storm intensity corresponding to a duration equal to the time of concentration (in/hr) and for a design frequency

PEAK DISCHARGE COMPUTATION

The Rational Method

A watershed is composed of the following land uses and runoff coefficients

Land Use	C_i	A_i (acres)
Open space	0.19	14.2
Forest	0.14	11.6
Residential (1/2 acre)	0.32	8.9
Light Comercial	0.89	4.3
Streets	0.82	3.9

The rainfall intensity is 3.6 in/hr. The peak discharge is

Land Use	C_i	A_i (acres)	Ci Ai
Open space	0.19	14.2	2.70
Forest	0.14	11.6	1.62
Residential (1/2 acre)	0.32	8.9	2.85
Light Comercial	0.89	4.3	3.83
Streets	0.82	3.9	3.20
	SUMMATION	42.9	14.20
Cave =	0.33		
Intensity (in/hr) =	3.60		
Area (acres) =	42.90		
Qp (cfs) =	**51.10**		

PEAK DISCHARGE COMPUTATION

The Rational Method: Multiple inlets

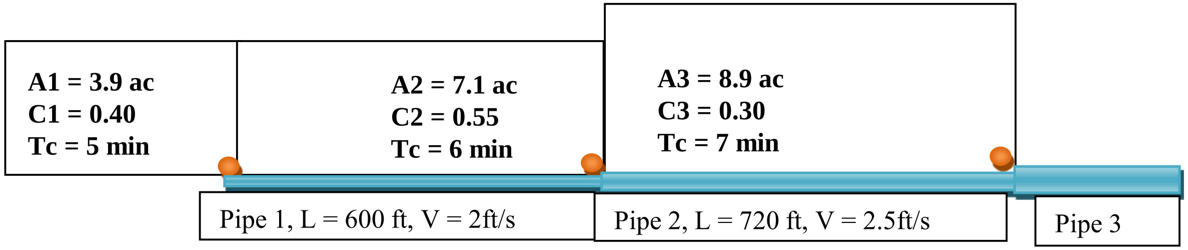
- Where a watershed is non-homogeneous and where multiple inlets and pipe systems are involved.

  - For each inlet área at the headwater of a drainage área the Rational method is used to compute the peadk discharge

  - For locations where water is arriving from two or more inlet areas, use THE LONGEST TIME OF CONCENTRATION to find the intensity. A weighted runoff coefficient is computed, and the total drainage area to that point is used.

PEAK DISCHARGE COMPUTATION
The Rational Method: Multiple inlets

The IDF curves for Civil City are given in the figure. Three areas are going to be drained by pipes as shown in the schematic. Use the Rational Method to obtain the design discharge for the pipe segments for a 50 yr return period.



A1 = 3.9 ac
C1 = 0.40
Tc = 5 min

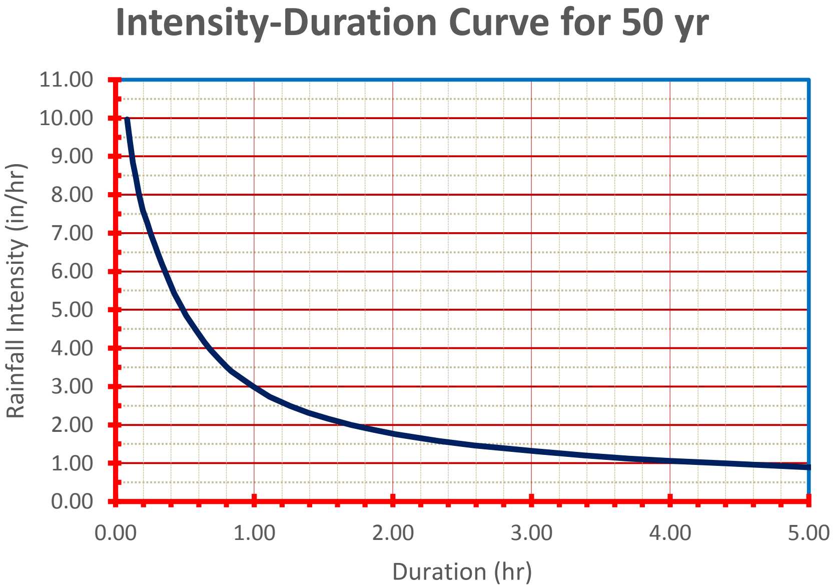
A2 = 7.1 ac
C2 = 0.55
Tc = 6 min

A3 = 8.9 ac
C3 = 0.30
Tc = 7 min

Pipe 1, L = 600 ft, V = 2ft/s

Pipe 2, L = 720 ft, V = 2.5ft/s

Pipe 3



The Rational Method

1) Compute the peak discharge at inlet 1:

$Q_{p1} = C_1\, i_1\, A_1$,   A in acres and *i* in in/hr

i_1 corresponds to a 5 min time of concentration

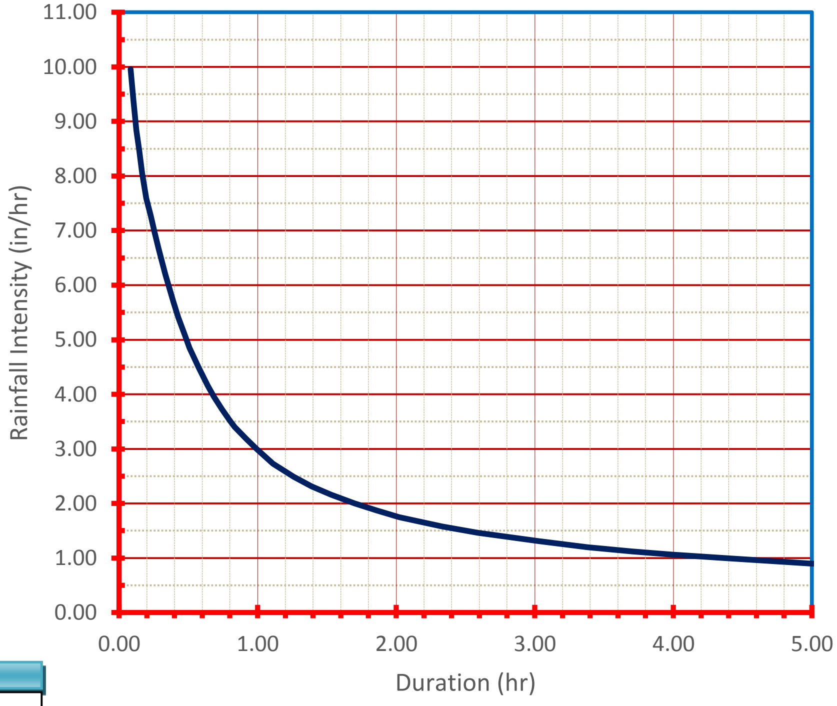
From IDF curve for 50 yrs and 5 min:

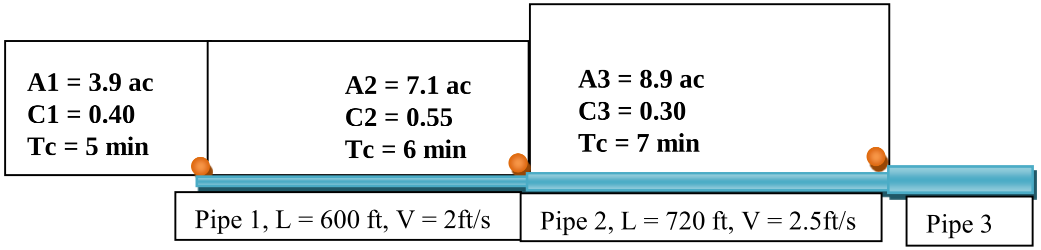
$$i_1 = 10 \text{ in/hr}$$

Design flow for pipe 1 would be:

$Q_{p1} = 0.4*10*3.9 = 15.6$ cfs

Intensity-Duration Curve for 50 yr





A1 = 3.9 ac
C1 = 0.40
Tc = 5 min

A2 = 7.1 ac
C2 = 0.55
Tc = 6 min

A3 = 8.9 ac
C3 = 0.30
Tc = 7 min

Pipe 1, L = 600 ft, V = 2ft/s

Pipe 2, L = 720 ft, V = 2.5ft/s

Pipe 3

The Rational Method

2) Compute peak discharge for PIPE 2:

It is common practice to compute the discharge for the total area of drainage using a weighted runoff coefficient and a rainfall intensity based on the longest time of concentration.

In this case there are two times of concentration to arrive at Inlet 2:

1) 5 min from A_1 + (600/(2 x 60)) min travel time inside pipe 1 = 10 min
2) 6 min time of concentration from A2

Choose the longest: T_{cpipe2} = 10 min

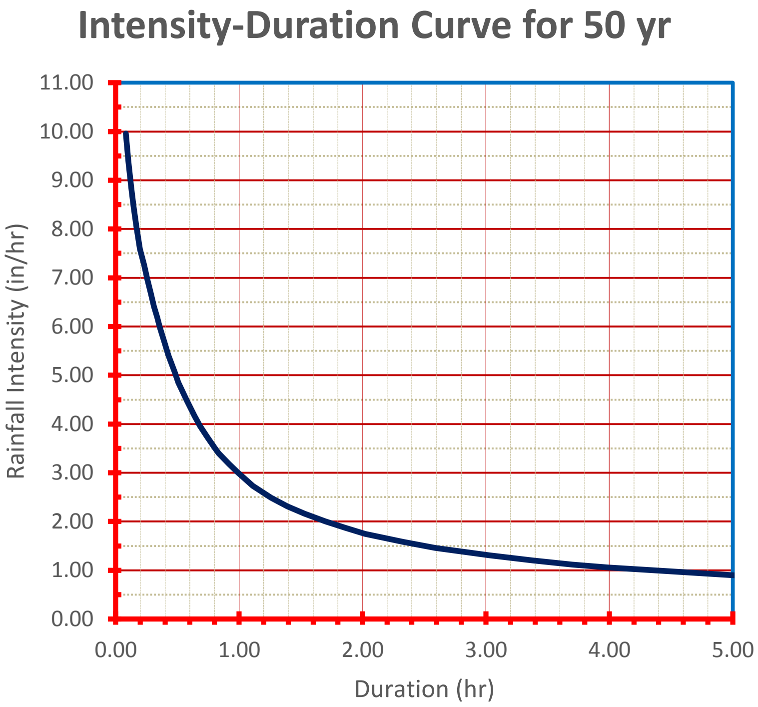
For this time the intensity from the IDF is *i* = 8.2 in/hr

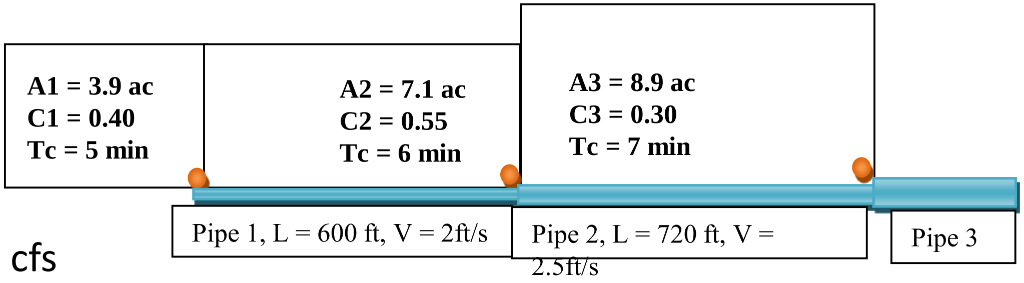
Use weighted C_w as:

C_w = (0.4 x 3.9 + 0.55 x 7.1)/(3.9+7.1) = 0.497

Use total area: $A_1 + A_2$ = 3.9 + 7.4 = 11.0 ac

Finally computed Q_{p2} = 0.497 x 8.2 x 11 = 44.8 cfs




A1 = 3.9 ac
C1 = 0.40
Tc = 5 min

A2 = 7.1 ac
C2 = 0.55
Tc = 6 min

A3 = 8.9 ac
C3 = 0.30
Tc = 7 min

Pipe 1, L = 600 ft, V = 2ft/s

Pipe 2, L = 720 ft, V = 2.5ft/s

Pipe 3

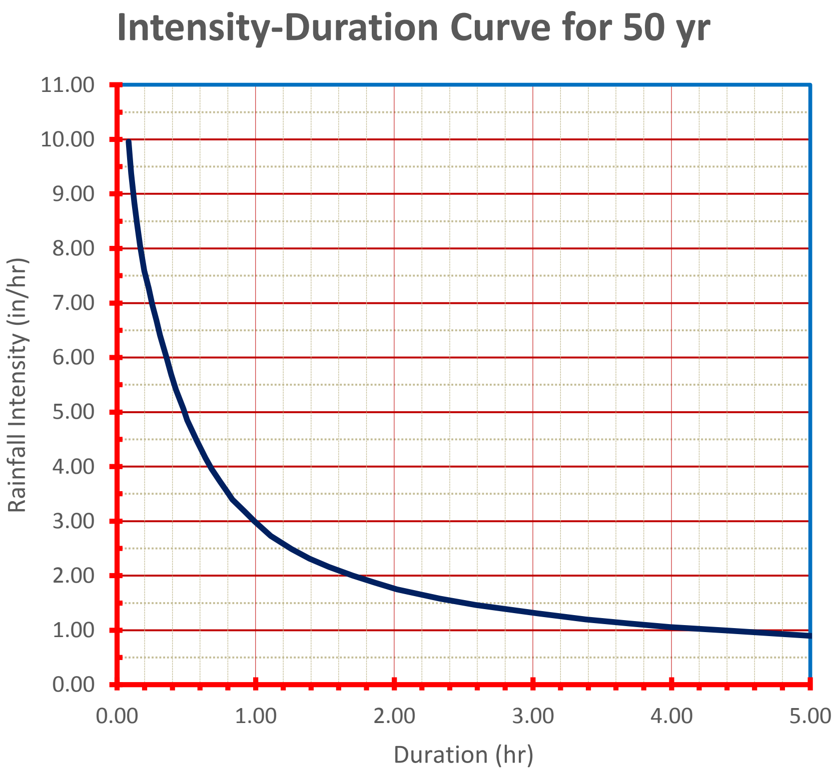
The Rational Method

3) Compute peak discharge for outlet pipe

There are 3 possible travel times:

a) 5 + 600/(2x60) + 720/(2.5x60) = 5 + 5 + 4.8 = 15 min
b) 6 + 720/(2.5 x 60) = 11 min
c) 7 min from A_3

Use the longest: 15 min.



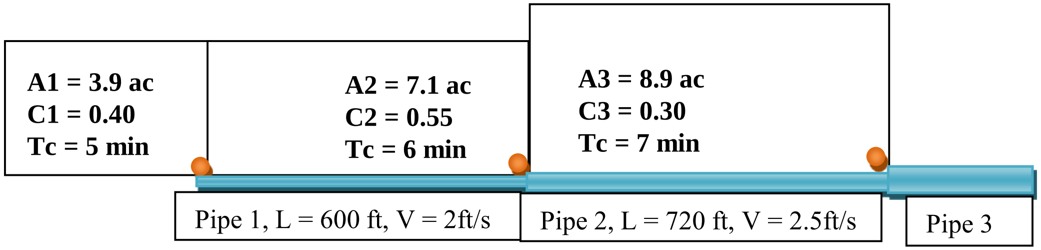
Get weighted Cw:

C_w = (0.4 x 3.9 + 0.55 x 7.1 + 0.3 x 8.9)/(3.9 + 7.1 + 8.0) = 0.409

A = 3.9 + 7.1 + 8.9 = 19.9 ac

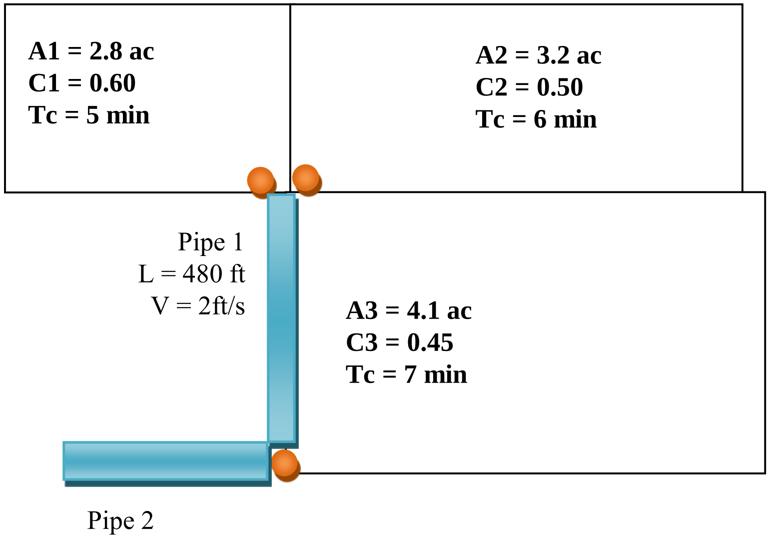
i_{15min} = 7.1 in/hr

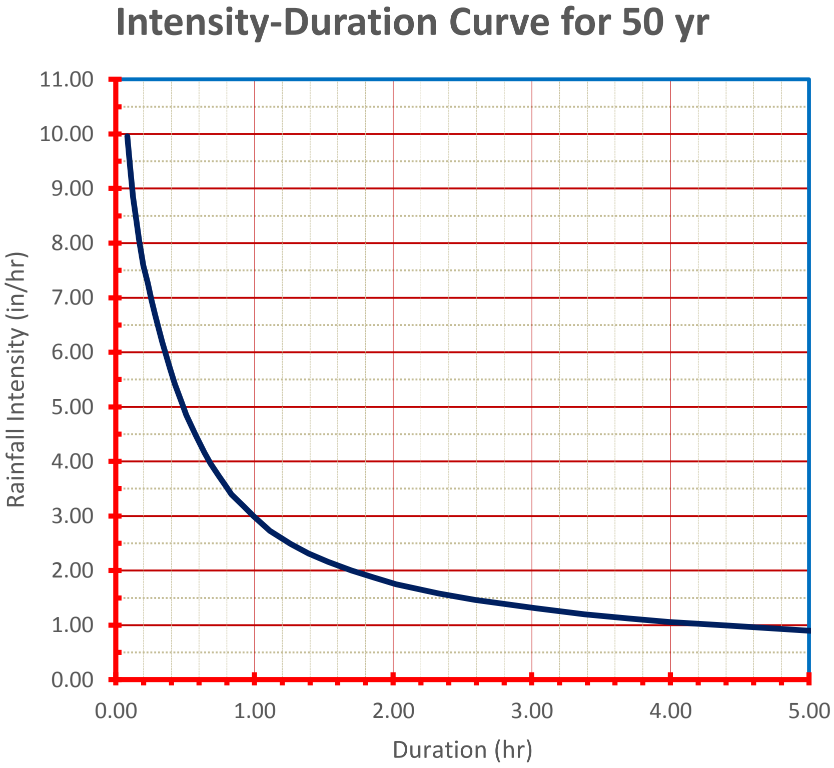
Q_{p3} = 0.409 x 7.1 x 19.9 = 57.8 cfs



The Rational Method

What is the peak discharge for the most downstream inlet using a 50 yr return period and the rational method is





The Rational Method

For Pipe 1:

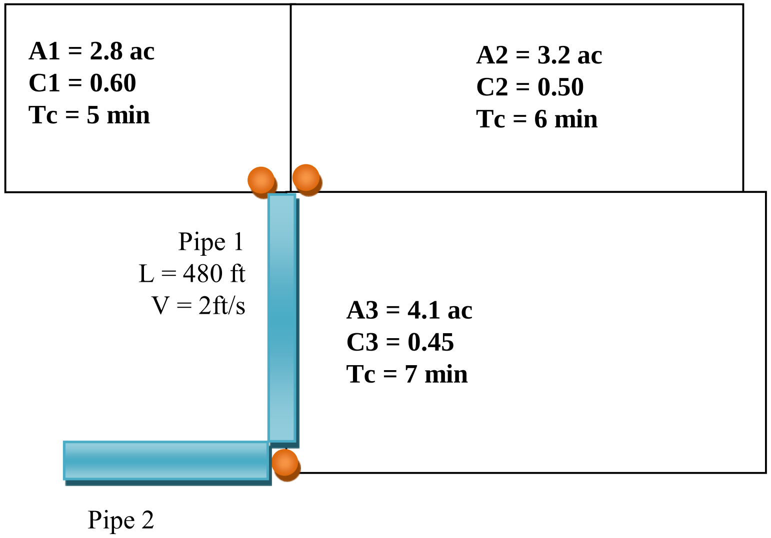
$A_T = 2.8 + 3.2 = 6.0$ ac

$C_w = (0.6 \times 2.8 + 0.5 \times 3.2)/(2.8+3.2) = 0.547$

$t_c = 6$ min

$i = 9.5$ in/hr

$Q_{p1} = 0.547 \times 9.5 \times 6 = 31.2$ cfs



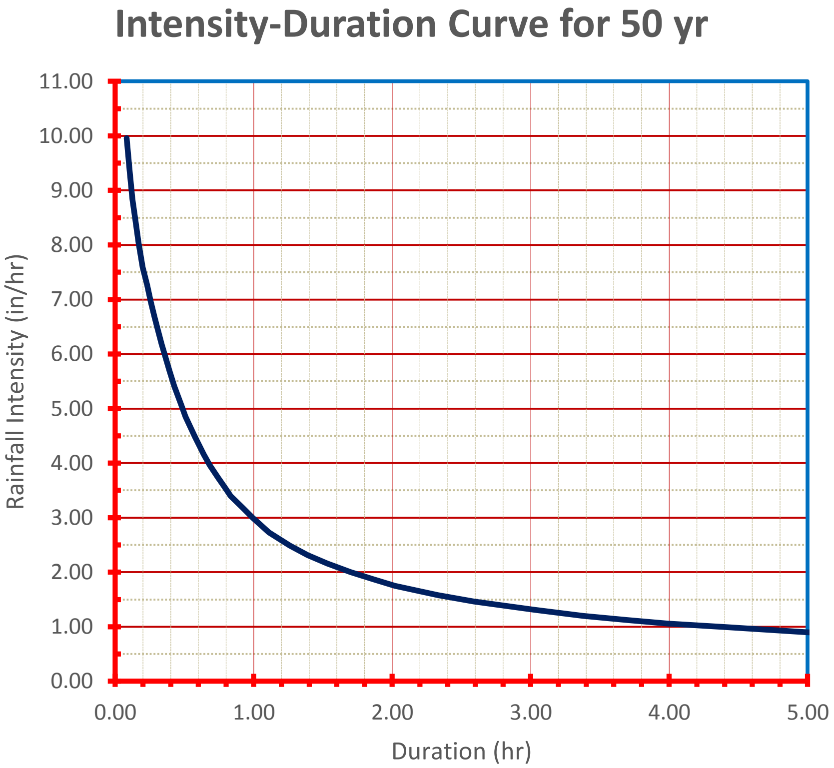
A1 = 2.8 ac
C1 = 0.60
Tc = 5 min

A2 = 3.2 ac
C2 = 0.50
Tc = 6 min

Pipe 1
L = 480 ft
V = 2ft/s

A3 = 4.1 ac
C3 = 0.45
Tc = 7 min

Pipe 2



The Rational Method

For Pipe 2:

A_T = 6.0 + 4.1 = 10.1 ac

C_w = (0.6 x 2.8 + 0.5 x 3.2 + 0.45 x 4.1)/(2.8 + 3.2 + 4.1) = 0.507

t_c = =6 + 480/(2 x 60) = 10 min

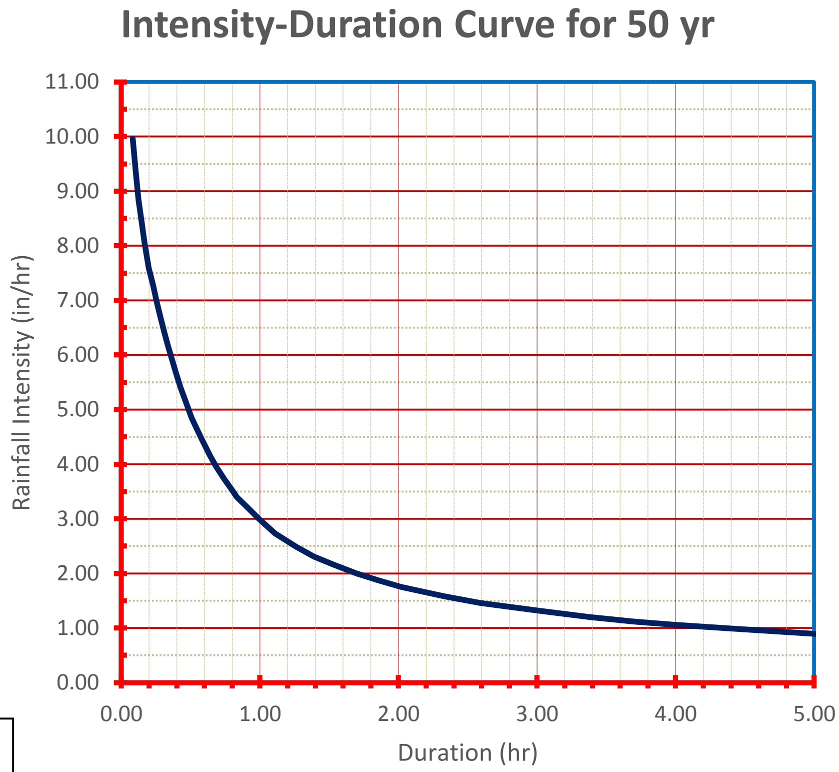
t_c = 5 + 480/(2 x 60) = 9 min

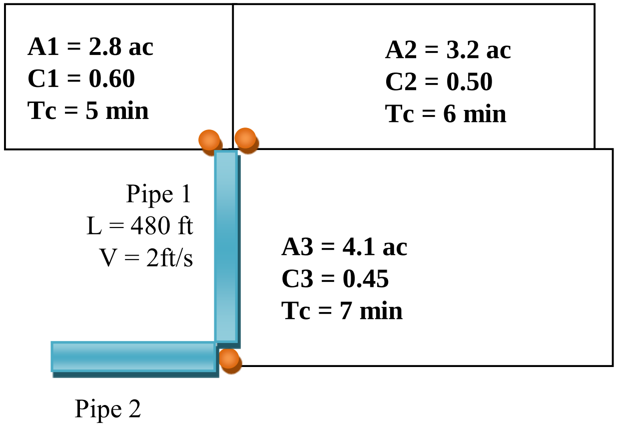
t_c = 7 min

Choose the longest: t_c = 10 min

The intensity for 10 min is : i = 8.2 in/hr; therefore:

Q_{p2} = 0.507 x 8.2 x 10.1 = 42 cfs





A1 = 2.8 ac
C1 = 0.60
Tc = 5 min

A2 = 3.2 ac
C2 = 0.50
Tc = 6 min

Pipe 1
L = 480 ft
V = 2ft/s

A3 = 4.1 ac
C3 = 0.45
Tc = 7 min

Pipe 2

Time of concentration and travel time

- Definitions of time of concentration
    1. Time to equilibrium of the catchment under a steady rainfall excess (i.e. when the outflow from the catchment equals the rainfall excess on the catchment.
    2. The time between the center of mass of rainfall excess and the inflection point on the recession of the direct runoff hydrograph
    3. Travel time of a wave to move from the hydraulically most distance point in the catchment to the outlet
    4. Travel time required for a particle of water to flow hydraulically from the most distant point in the watershed to the outlet or design point

Travel Time

- Velocity Method
- The velocity is a function of the type of flow
  - Sheet flow
  - Concentrated flow
  - Gully flow
  - Channel flow or
  - Pipe flow
- Flow velocities are usually computed using a modified Manning’s equation

Travel Time

- Sheet flow Travel
  - Valid at the upper reaches of the watershed
  - Flow over surfaces at uniform, shallow depths
  - The distance from the upper end of the watershed to the point where significant concentrated flow begins is the sheet-flow length
  - For steep, impervious slopes it could be several hundred feet.
  - For shallow slopes or pervious surfaces could be shorter

Travel Time

- Sheet flow Travel
  - The Kinematic wave theory has been used to obtain the following equation for sheet flow travel time:

$$T_t = \frac{0.938}{i^{0.4}}\left(\frac{nL}{\sqrt{S}}\right)^{n}$$

Requires iterations because intensity is unknown

Approximation:

$$T_t = \frac{0.42}{P_2^{0.5}}\left(\frac{nL}{S^{0.5}}\right)^{0.8}$$

T_t = travel time in sheet-flow in minutes

L = flow length (ft)

S = average surface slope (ft/ft)

i = rainfall intensity for the time of concentration (in/hr)

P_2 = 2yr- 24 hr rainfall depth (in)

Use the values of n corresponding to overland flow

Travel Time

- Sheet flow Travel
  - Use IDF curves to obtain P2.
  - This equation can give long times of concentration
  - Limits are from 100 ft to 300 ft (100 ft is better!)
  - McCuen recommends to use the equation for a length, in feet, of:

$$100\, S^{0.5} n^{-1}$$

Travel Time

- Velocity Method
  - Based on the concept that the travel time T_t for a a particular flow path is a function of the length of flow (L) and the velocity (V).
  - $T_t = L/(60V)$
  - Where T_t is the travel time in minutes for L and V in consistent units system (time in seconds)
  - The travel time is computed for the principal flow path.

Travel Time

- Velocity Method
  - Where the principal flow path consists of segments with different slope, land cover and flow behavior it must be divided in segments.
  - The time of concentration is the sum of the travel times:

$$t_c = \sum_{i=1}^{k} T_{ti} = \sum_{i=1}^{k} \left(\frac{L_i}{60V_i}\right)$$

Travel Time

- Estimation of Velocities for given Manning’s n and Hydraulic Radius
  - For given n value and surface roughness the Manning’s equation reduces to:

$$V = kS^{0.5}$$

  - Where

$$k = 1.486R_h^{2/3}/n$$

  - Since the hydraulic radius changes with time during the storm, this simplification is subject to significant errors.
  - Note that the value of Manning’s n for overland flow is not the same as the for open channel.

Travel Time

- Estimation of Velocities for given Manning’s n and Hydraulic Radius using MODIFIED MANNING’S EQ.

For given n value and surface roughness the Manning’s equation reduces to:

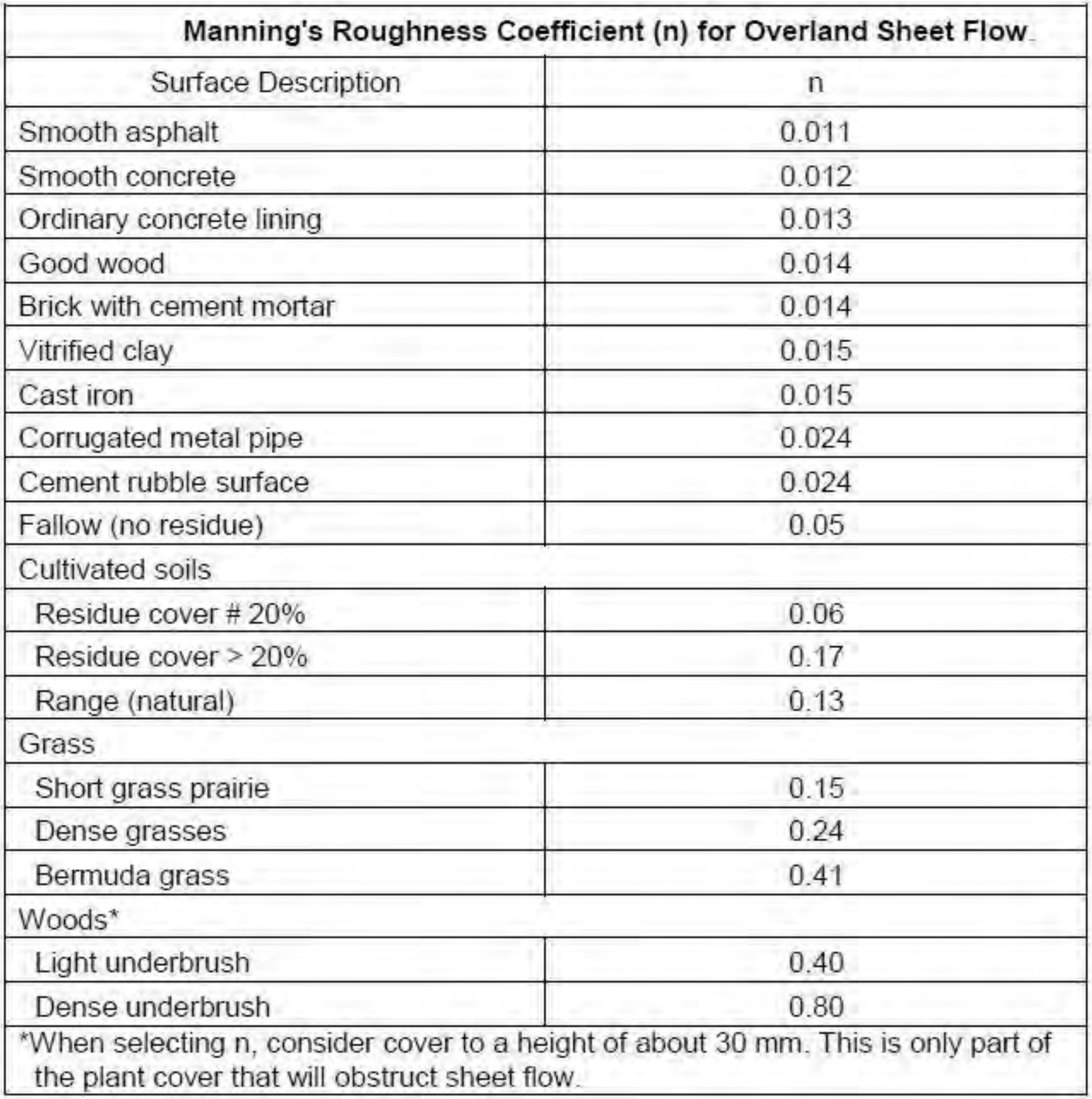
$$V = kS^{0.5}$$

Where

$$k = 1.486R_h^{2/3}/n$$

Since the hydraulic radius changes with time during the storm, this simplification is subject to significant errors.

The value of Manning’s n for overland flow is not the same as the for open channel.



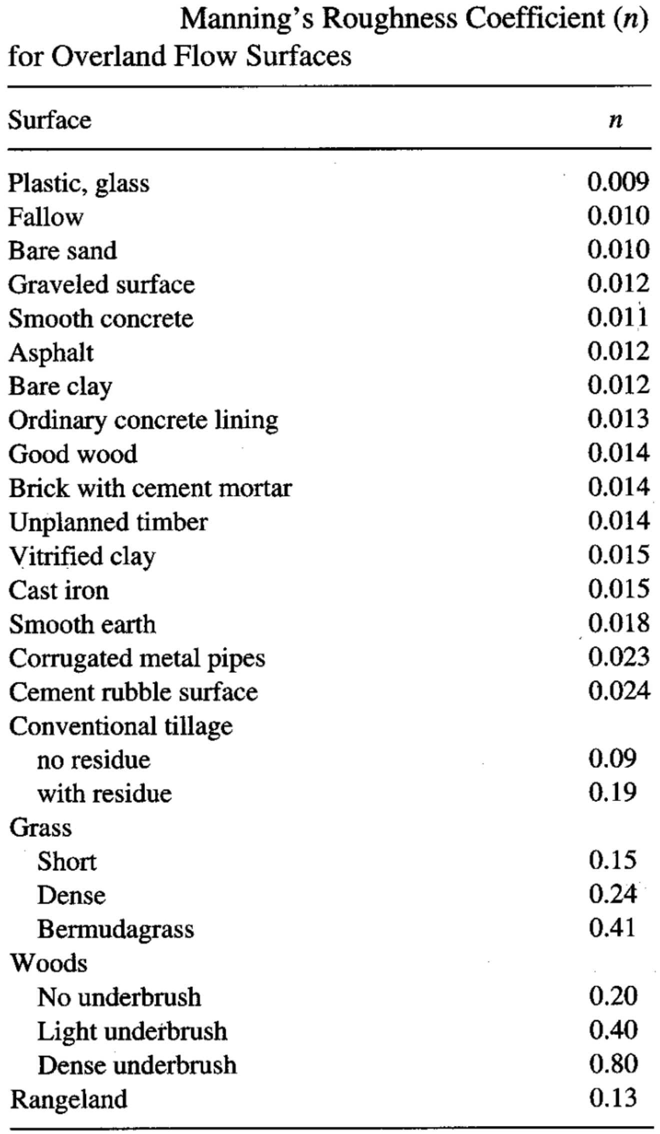
Manning's Roughness Coefficient (n) for Overland Sheet Flow.

Surface Description	n
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Good wood	0.014
Brick with cement mortar	0.014
Vitrified clay	0.015
Cast iron	0.015
Corrugated metal pipe	0.024
Cement rubble surface	0.024
Fallow (no residue)	0.05
Cultivated soils	
Residue cover # 20%	0.06
Residue cover > 20%	0.17
Range (natural)	0.13
Grass	
Short grass prairie	0.15
Dense grasses	0.24
Bermuda grass	0.41
Woods*	
Light underbrush	0.40
Dense underbrush	0.80

*When selecting n, consider cover to a height of about 30 mm. This is only part of the plant cover that will obstruct sheet flow.

Travel Time

- Manning’s coefficients for Overland flow

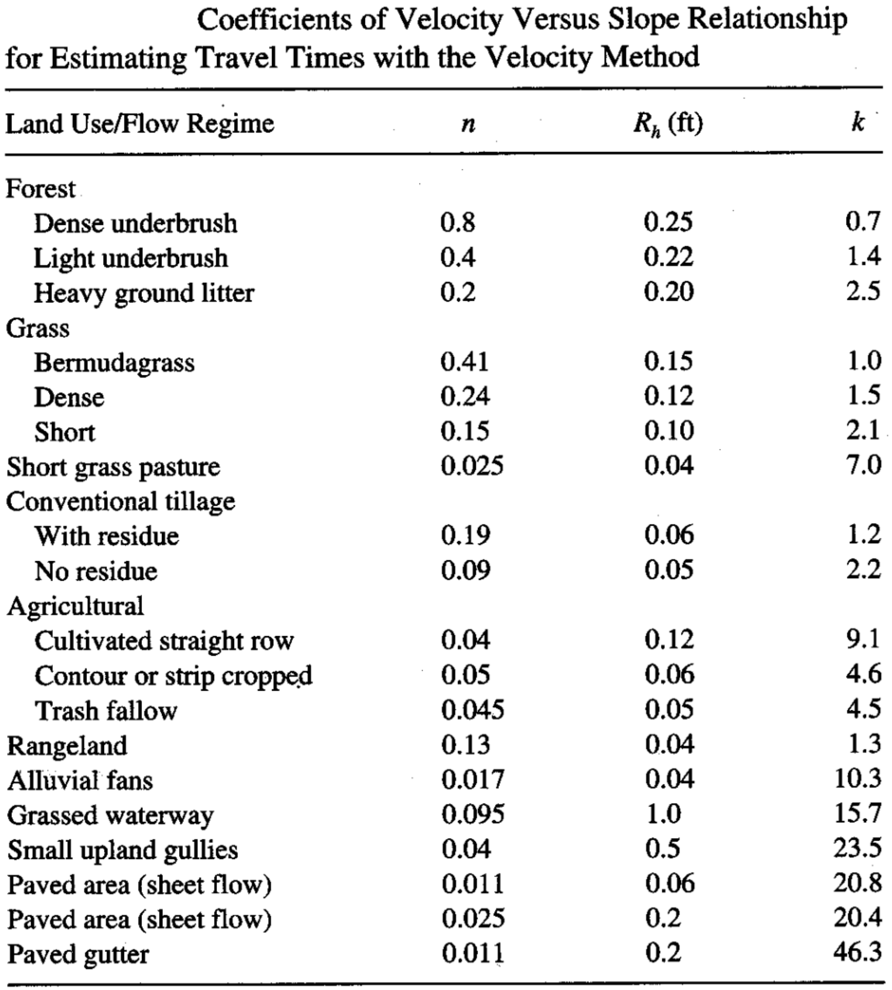


Manning’s Roughness Coefficient (*n*) for Overland Flow Surfaces

Surface	*n*
Plastic, glass	0.009
Fallow	0.010
Bare sand	0.010
Graveled surface	0.012
Smooth concrete	0.011
Asphalt	0.012
Bare clay	0.012
Ordinary concrete lining	0.013
Good wood	0.014
Brick with cement mortar	0.014
Unplanned timber	0.014
Vitrified clay	0.015
Cast iron	0.015
Smooth earth	0.018
Corrugated metal pipes	0.023
Cement rubble surface	0.024
Conventional tillage	
no residue	0.09
with residue	0.19
Grass	
Short	0.15
Dense	0.24
Bermudagrass	0.41
Woods	
No underbrush	0.20
Light underbrush	0.40
Dense underbrush	0.80
Rangeland	0.13

Travel Time

- Coefficients for estimating travel time in Sheet flow using Manning’s simplified equation

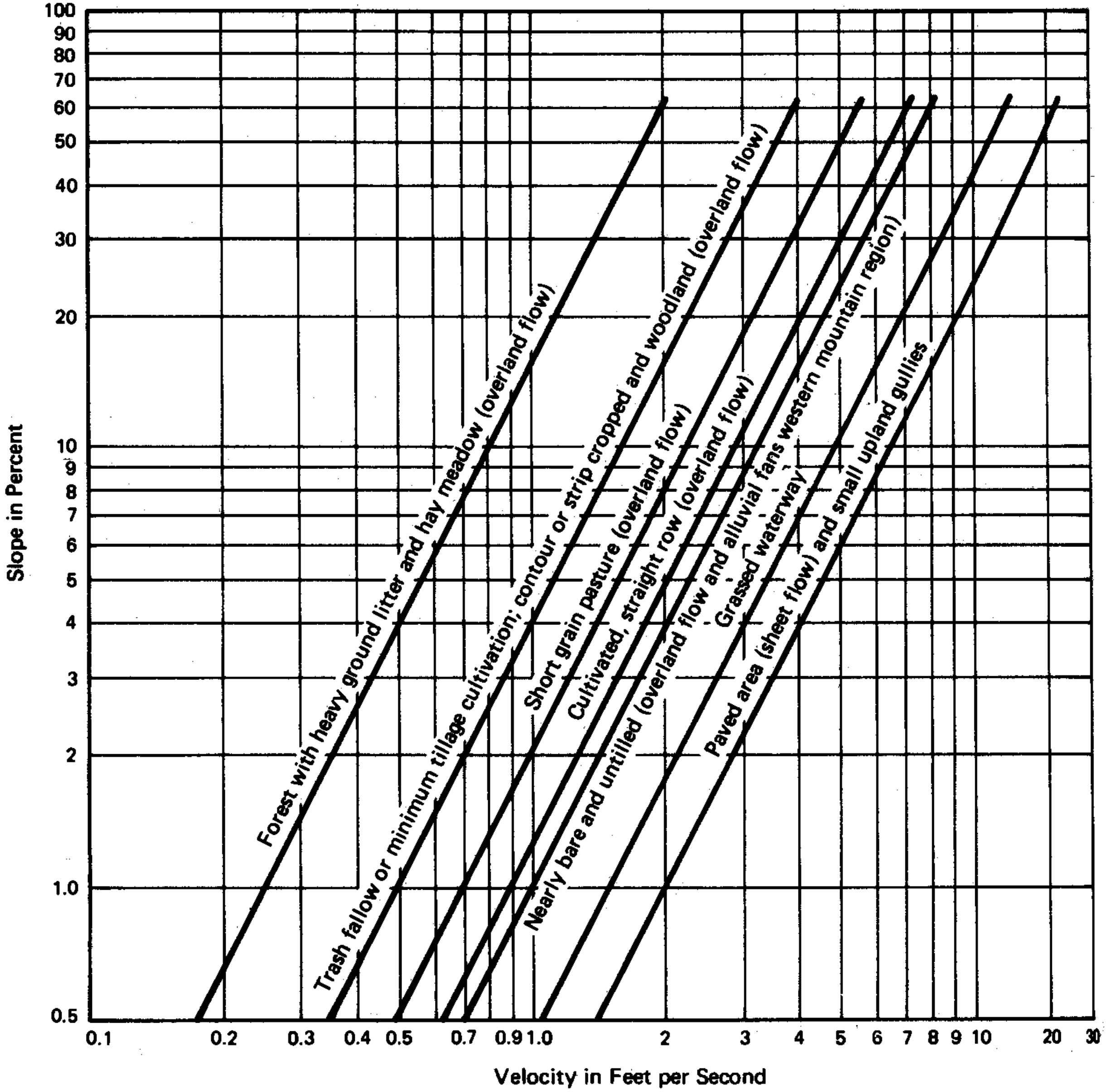


Coefficients of Velocity Versus Slope Relationship for Estimating Travel Times with the Velocity Method

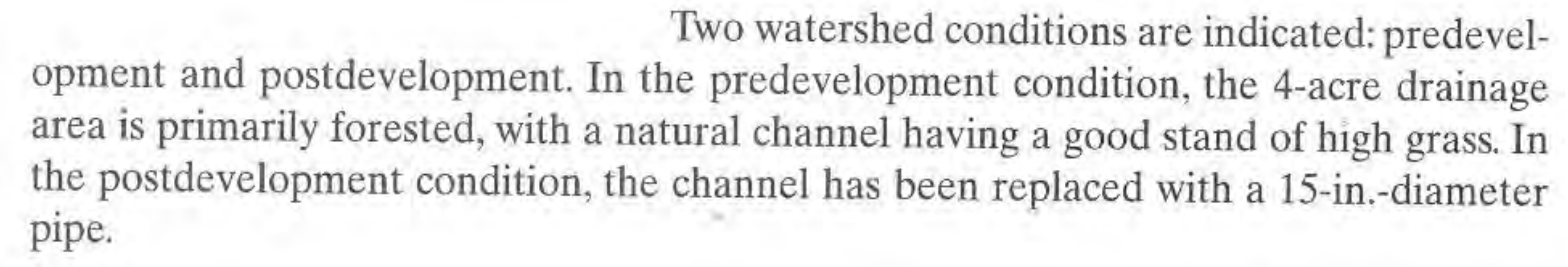
Land Use/Flow Regime	*n*	R_h (ft)	*k*
Forest			
Dense underbrush	0.8	0.25	0.7
Light underbrush	0.4	0.22	1.4
Heavy ground litter	0.2	0.20	2.5
Grass			
Bermudagrass	0.41	0.15	1.0
Dense	0.24	0.12	1.5
Short	0.15	0.10	2.1
Short grass pasture	0.025	0.04	7.0
Conventional tillage			
With residue	0.19	0.06	1.2
No residue	0.09	0.05	2.2
Agricultural			
Cultivated straight row	0.04	0.12	9.1
Contour or strip cropped	0.05	0.06	4.6
Trash fallow	0.045	0.05	4.5
Rangeland	0.13	0.04	1.3
Alluvial fans	0.017	0.04	10.3
Grassed waterway	0.095	1.0	15.7
Small upland gullies	0.04	0.5	23.5
Paved area (sheet flow)	0.011	0.06	20.8
Paved area (sheet flow)	0.025	0.2	20.4
Paved gutter	0.011	0.2	46.3

Travel Time

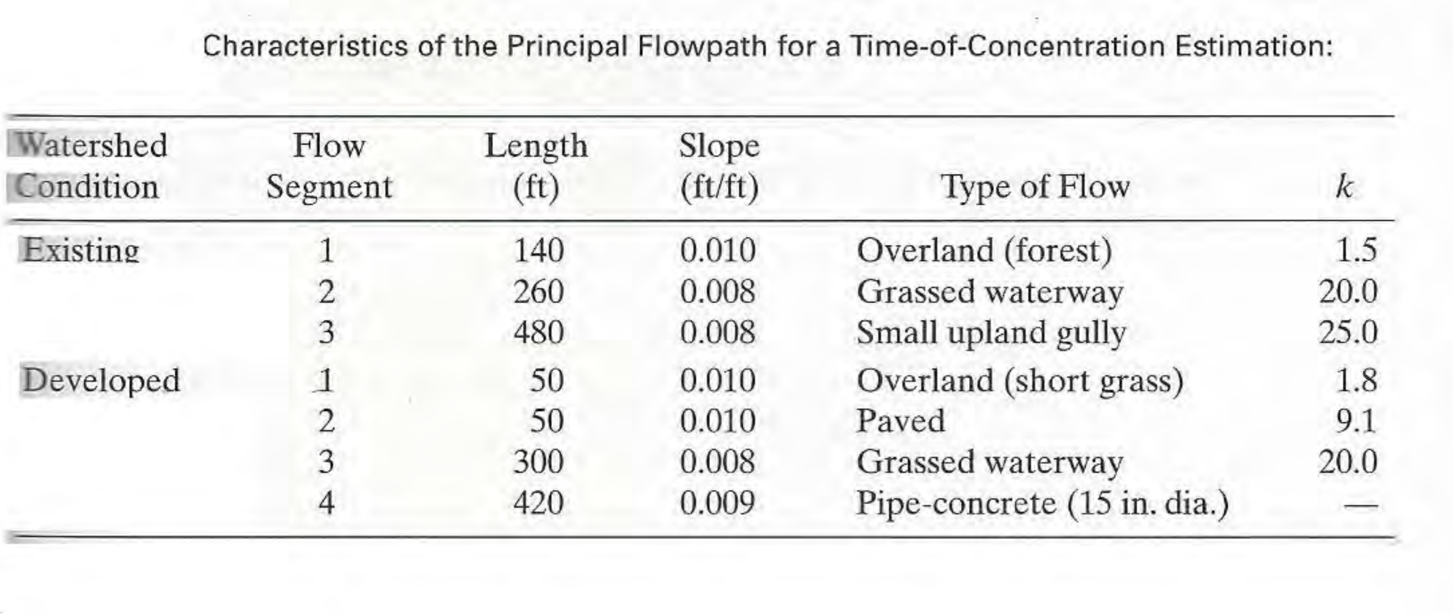
- Velocities for the upland method of estimating travel time



Example: Calculation of Travel time



Two watershed conditions are indicated: predevelopment and postdevelopment. In the predevelopment condition, the 4-acre drainage area is primarily forested, with a natural channel having a good stand of high grass. In the postdevelopment condition, the channel has been replaced with a 15-in.-diameter pipe.



Characteristics of the Principal Flowpath for a Time-of-Concentration Estimation:

Watershed Condition	Flow Segment	Length (ft)	Slope (ft/ft)	Type of Flow	*k*
Existing	1	140	0.010	Overland (forest)	1.5
	2	260	0.008	Grassed waterway	20.0
	3	480	0.008	Small upland gully	25.0
Developed	1	50	0.010	Overland (short grass)	1.8
	2	50	0.010	Paved	9.1
	3	300	0.008	Grassed waterway	20.0
	4	420	0.009	Pipe-concrete (15 in. dia.)	—

Adapted from Mc.Cuen 2004

Example: Calculation of Travel time

- For the slopes given and using the modified Manning's equation:

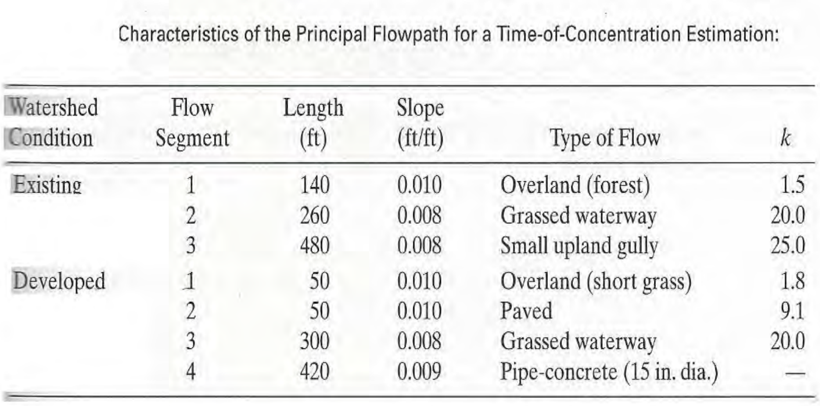
$$V = kS^{0.5}$$

- The velocities are:

$V_{forest} = 1.5\text{x}0.01^{0.5} = 0.15$ ft/s

$V_{grass} = 20\text{x}0.008^{0.5} = 1.79$ ft/s

$V_{gully} = 25\text{x}0.008^{0.5} = 2.24$ ft/s



Characteristics of the Principal Flowpath for a Time-of-Concentration Estimation:

Watershed Condition	Flow Segment	Length (ft)	Slope (ft/ft)	Type of Flow	*k*
Existing	1	140	0.010	Overland (forest)	1.5
	2	260	0.008	Grassed waterway	20.0
	3	480	0.008	Small upland gully	25.0
Developed	1	50	0.010	Overland (short grass)	1.8
	2	50	0.010	Paved	9.1
	3	300	0.008	Grassed waterway	20.0
	4	420	0.009	Pipe-concrete (15 in. dia.)	—

- The predevelopment time of concentration is:

$$t_c = \frac{140}{0.25} + \frac{260}{1.4} + \frac{480}{2.1} = 560 + 186 + 229 = 975\ sec = 16.2 \text{ min}$$

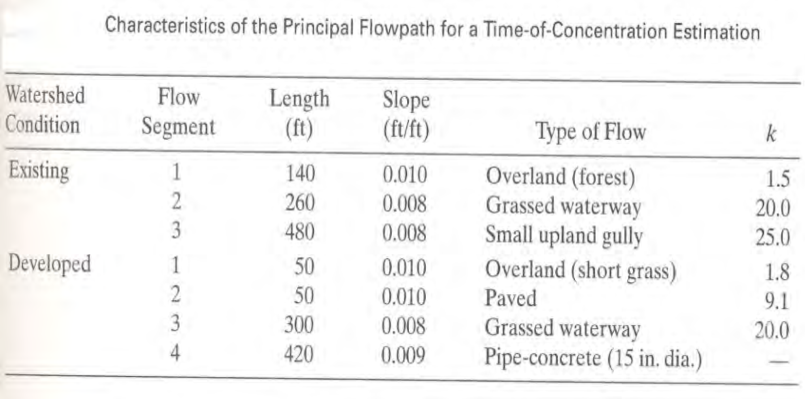
Example: Calculation of Travel time

- Using the same equation for the overland grass paved and grass waterway sections:
- The velocities are:

$V_{ovgrass} = 1.8x0.01^{0.5} = 0.18$ ft/s

$V_{Paved} = 9.1x0.01^{0.5} = 0.91$ ft/s

$V_{grass} = 20x0.008^{0.5} = 1.79$ ft/s



Characteristics of the Principal Flowpath for a Time-of-Concentration Estimation

Watershed Condition	Flow Segment	Length (ft)	Slope (ft/ft)	Type of Flow	*k*
Existing	1	140	0.010	Overland (forest)	1.5
	2	260	0.008	Grassed waterway	20.0
	3	480	0.008	Small upland gully	25.0
Developed	1	50	0.010	Overland (short grass)	1.8
	2	50	0.010	Paved	9.1
	3	300	0.008	Grassed waterway	20.0
	4	420	0.009	Pipe-concrete (15 in. dia.)	—

- For the concrete pipe assume n = 0.011 and full pipe velocity. Use Manning's equation:

$$V = \frac{1.486}{0.011}\left(\frac{1.25}{4}\right)^{0.67}(0.009)^{0.5} = 5.9 \text{ ft/sec}$$

- The time of concentration for post development conditions is:

$$t_c = \frac{50}{0.21} + \frac{50}{2.08} + \frac{300}{1.40} + \frac{420}{5.9}$$

$$= 238 + 24 + 214 + 71 = 547 \text{ sec} = 9.1 \text{ min}$$

SCS (NRCS) Rainfall-Runoff Depth Relation

- According to the NRCS method the runoff (in inches) can be computed as:

$$Q = (P-0.2S)^2/(P+0.8S)$$

- Empirical evidence indicated that: $I_a = 0.2\ S$

$$S = \text{Potential maximum retention}$$

- Empirical analysis led to

$$S = 1000/CN - 10$$

*where **CN** is the **CURVE NUMBER**. This is an index that represents the combination of a hydrologic soil group and a land use and treatment class.*

- This equation has the following limitation: $P >= 0.2S$
- When $P < 0.2S$ then $Q = 0$

SCS (NRCS) Rainfall-Runoff Depth Relation

Cn is a function of

1. Hydrologic soil group (HSG)
2. Cover type
3. Treatment
4. Hydrologic condition
5. Antecedent runoff condition

Hydrologic soil groups

- Infiltration rates of soils vary widely and are affected by subsurface permeability as well as surface intake rates.
- Soils are classified into four HSG's (A, B, C, and
- D) according to their minimum infiltration rate
- is obtained for bare soil after prolonged wetting.

Hydrologic soil groups

- **Group A** *soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sand or gravel and have a high rate of water transmission*
- *(greater than 0.30 in/hr).*
- **Group B** *soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (0.15-0.30 in/hr).*
- **Group C** *soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (0.05-0.15 in/hr).*
- **Group D** *soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0-0.05 in/hr).*

TR-55, Second Ed., June 1986

Cover type – Treatment-Hydrologic Condition

- **Cover type** indicates the soil situation: bare soil, impervious surfaces or vegetation. Requires field reconnaissance, aerial photos, and land use maps
- **Treatment** modifies the cover type. Refers to management of agricultural lands. Includes practices such as contouring, terracing, crop rotations and tillage
- **Hydrologic Condition** indicates the effects of cover type and treatment on infiltration and runoff.
- Good hydrologic condition indicates that the soil usually has a low runoff potential for that specific soil group

Antecedent Moisture Condition

- **Condition I:** Soils are dry but not to wilting point; satisfactory cultivation taken place
- **Condition II:** Average condition
- **Condition III:** Heavy rainfall, or light rainfall and low temperatures have occurred within the last five days; saturated soil.

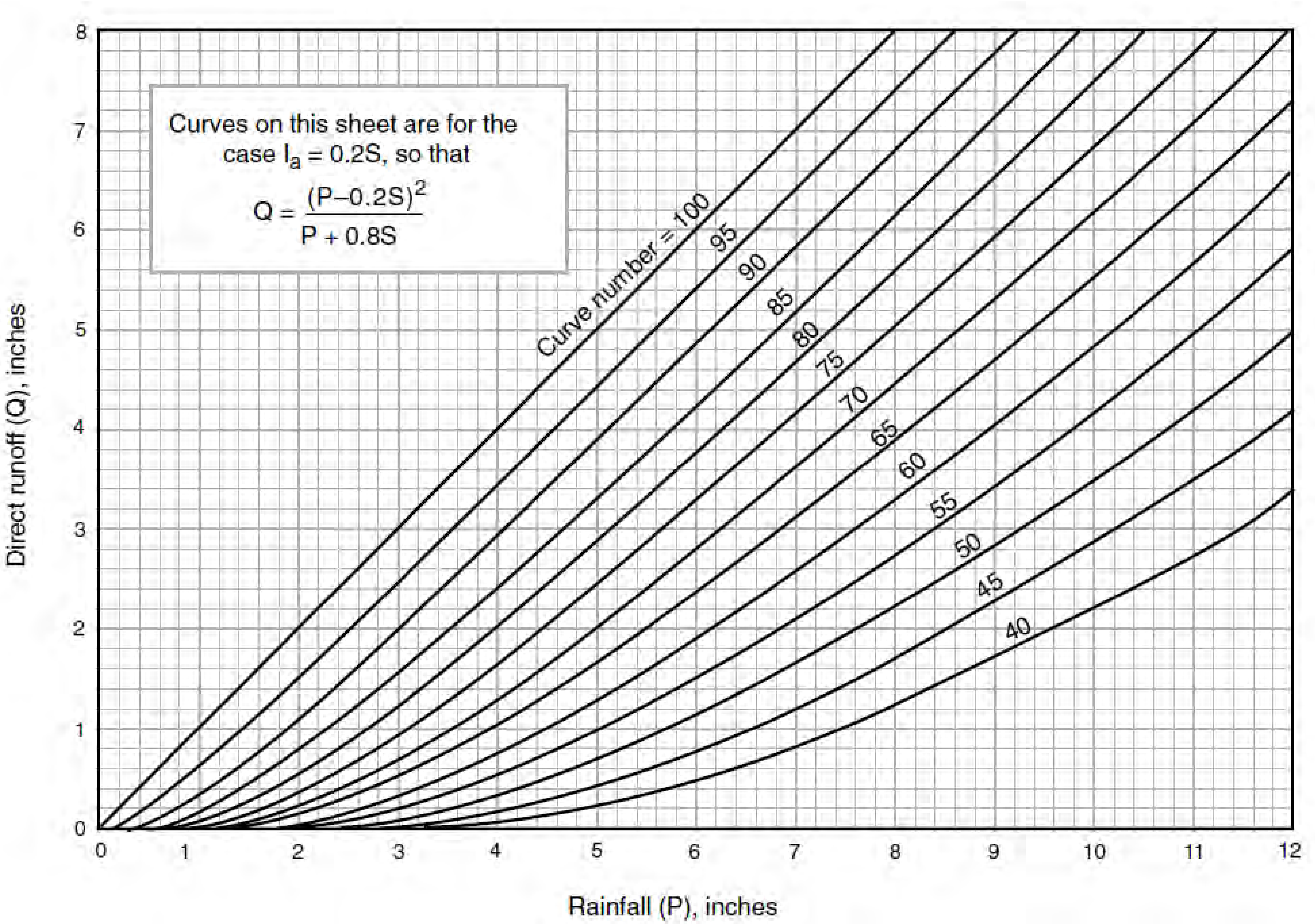
		HYDROLOGIC SOIL GROUP			
LAND USE DESCRIPTION		A	B	C	D
Cultivated land[1]					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow					
Good condition		30	58	71	78
Wood or forest land					
Thin stand, poor cover, no mulch		45	66	77	83
Good cover[2]		25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.					
Good condition: grass cover on 75% or more of the area		39	61	74	80
Fair condition: grass cover on 50–75% of the area		49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial districts (72% impervious)		81	88	91	93
Residential[3]					
Average lot size	Average % impervious[4]				
1/8 ac or less	65	77	85	90	92
1/4 ac	38	61	75	83	87
1/3 ac	30	57	72	81	86
1/2 ac	25	54	70	80	85
1 ac	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5]		98	98	98	98
Streets and roads					
Paved with curbs and storm sewers[6]		98	98	98	98
Gravel		76	85	89	91
Dirt		72	82	87	89

Curve Number for Condition II	Corresponding Curve Number for Condition:	
	I	III
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	7	33
10	4	26
5	2	17
0	0	0

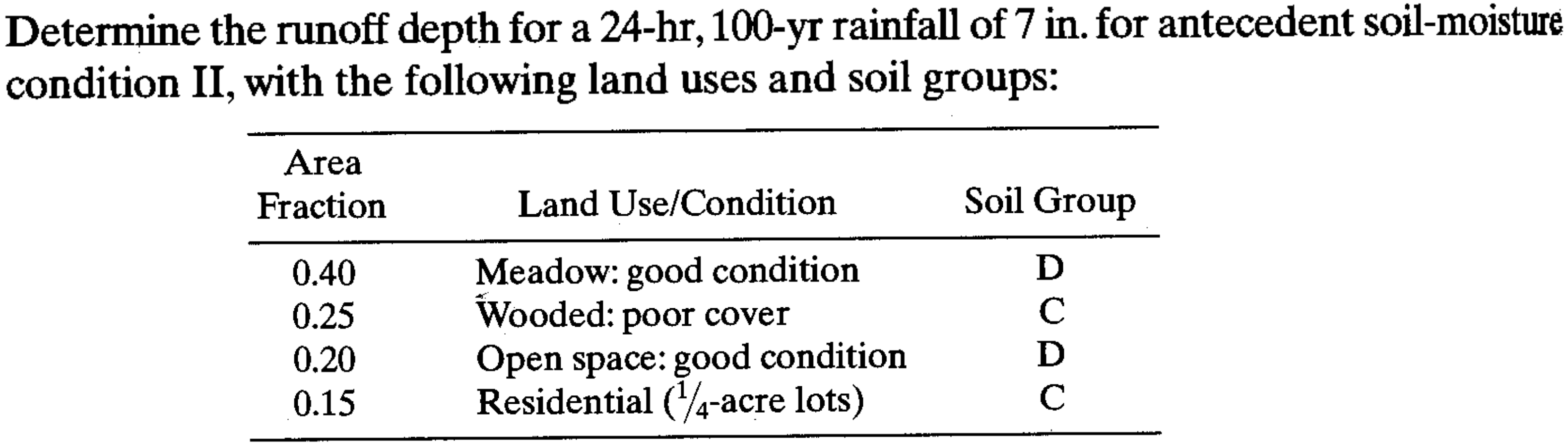
Source: U.S. Soil Conservation Service (1972).

Runoff Depth Estimation

- Graphical Solution of SCS Runoff Equation



Runoff Depth Estimation using CN



Determine the runoff depth for a 24-hr, 100-yr rainfall of 7 in. for antecedent soil-moisture condition II, with the following land uses and soil groups:

Area Fraction	Land Use/Condition	Soil Group
0.40	Meadow: good condition	D
0.25	Wooded: poor cover	C
0.20	Open space: good condition	D
0.15	Residential ($^{1}/_{4}$-acre lots)	C

- Get the corresponding CN from Table: 78, 77, 80 and 83
- Obtain the weighted **CN** :

$$CN = 0.40(78) + 0.25(77) + 0.20(80) + 0.15(83) = 78.9(\text{use } 79)$$

- For CN = 79 and 7 in rainfall the runoff depth is 4.59 in

TR-55 Peak Discharge Estimation

- Inputs:
  1. Tc (hr)
  2. Drainage Area (mi^2)
  3. Appropriate Rainfall Distribution (I, IA, II, or III)
  4. 24-hour Rainfall (in)
  5. CN

Presentation: “An Introduction to TR-55” by Kristina Schneider, February 27, 2001

TR-55 Peak Discharge Estimation

- Allows you to calculate the peak discharge
- Equation:

$$q_p = q_u A_m Q F_p$$

q_p = peak discharge (cfs)

q_u = unit peak discharge (csm/in)

A_m = drainage area (mi^2)

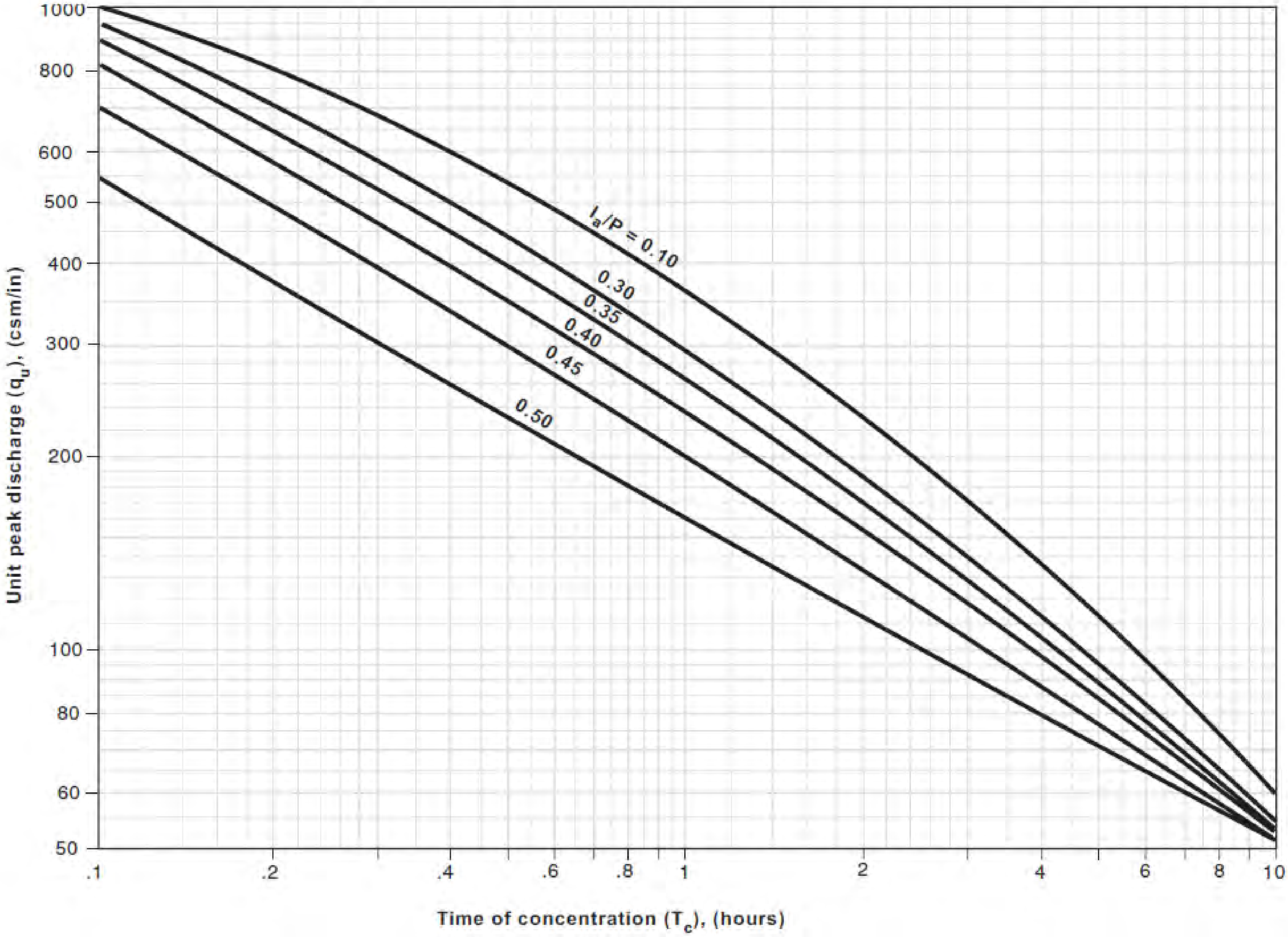
Q = runoff (in)

F_p = pond and swamp adjustment factor

Presentation: "An Introduction to TR-55" by Kristina Schneider, February 27, 2001

Graphical Solution

Exhibit 4-II Unit peal discharge (q_u) for NRCS (SCS) type II rainfall distribution



TR-55 Peak Discharge Estimation

Given: *The following physical and hydrologic conditions.*

- *3.3 sq km (1.27 mi2) of fair condition open space and 2.8 sq km (1.08 mi2)of large lot residential*
- *Negligible pond and swamp land*
- *Hydrologic soil type C*
- *Average antecedent moisture conditions*
- *Time of concentration is 0.8 hr*
- *24-hour, type II rainfall distribution, 10-year rainfall of 150 mm (5.9 in)*

TR-55 Peak Discharge Estimation

- *Step 1: Calculate the composite curve number*

  CN = Σ (CNx Ax)/A = [1.27(79) + 1.08(77)]/(1.27 + 1.08) = 78

- *Step 2: Calculate the retention S:*

  S = 1.0(1000/CN - 10) = 1.0[(1000/78) - 10] = 2.82 in

- *Step 3: Calculate the depth of direct runoff*

  $Q_D = (P-0.2S_R)^2 / (P+0.8S_R) = [5.9 - 0.2(2.82)]^2/[5.9 + 0.8(2.82)] = 3.49$in

- *Step 4: Determine Ia/P from equation Ia = 0.2S*

  $I_a = 0.2\ (2.82) = 0.564$

  $I_a/P = 0.564/5.9 = 0.096$ say 0.10

TR-55 Peak Discharge Estimation

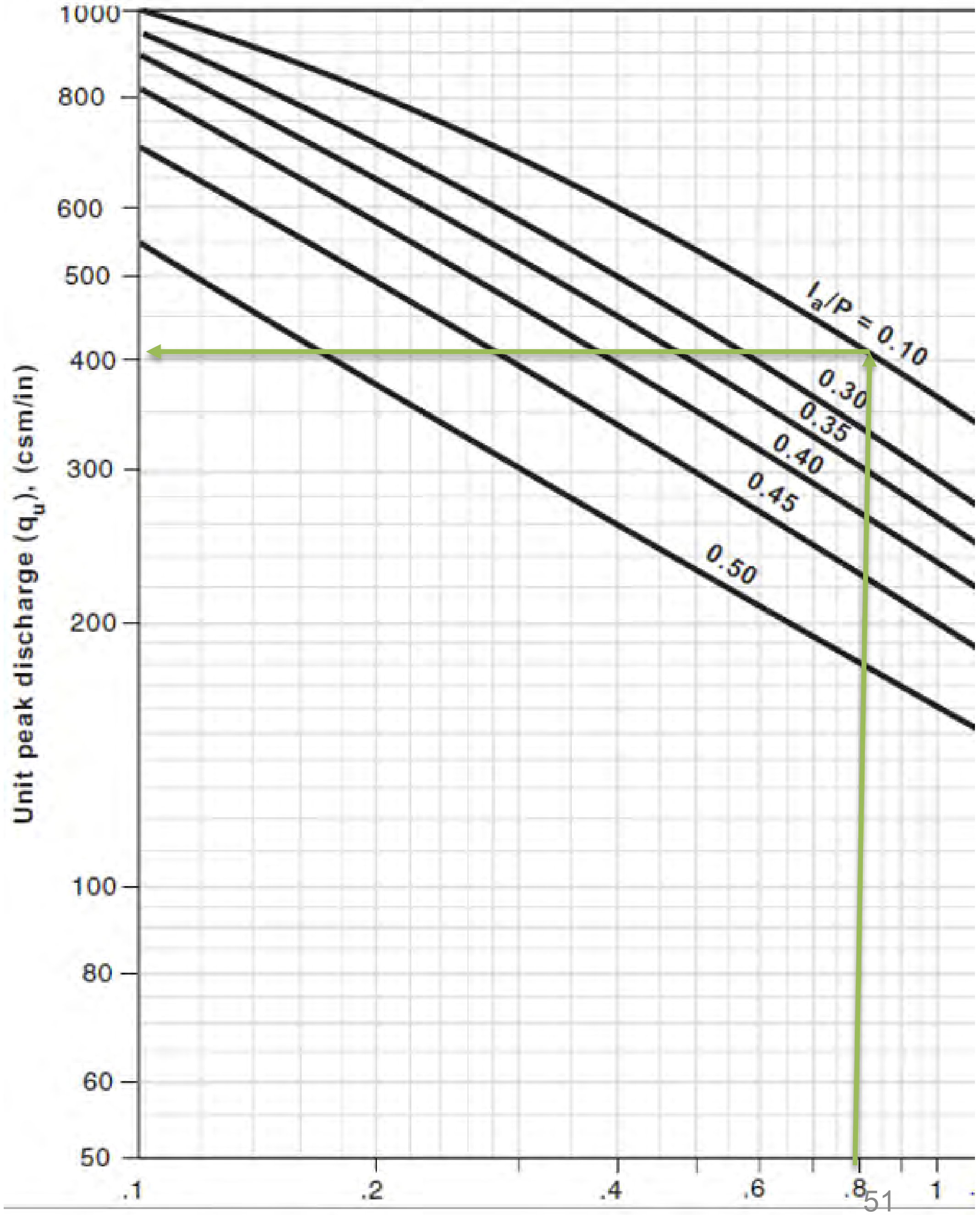
Step 5: Enter the graph to determine Unit Peak Flow

q_u = 410 ft^3/s/mi^2/in

Step 6: Calculate peak flow

$q_p = q_u A_k Q_D$ = (409) (2.35) (3.49)

q_p = 3,354 ft$_3$/s



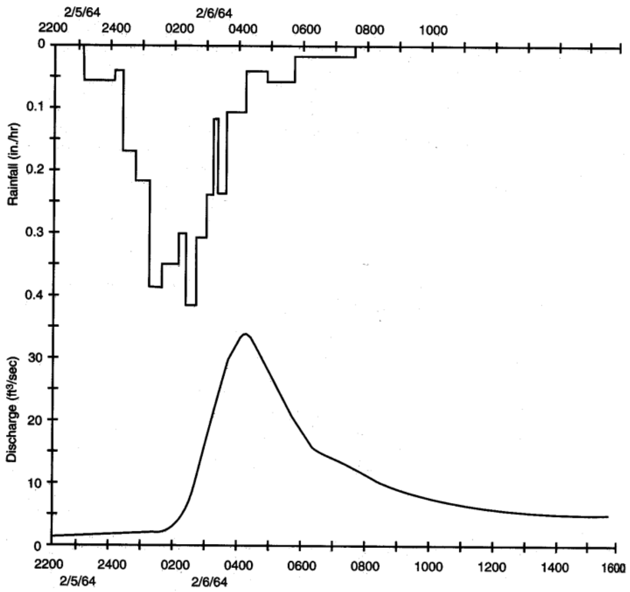
Hydrographs Analysis

- Base Flow Separation
  - Constant Discharge
  - Constant Slope
  - Concave Base flow separation
- Separation of losses
  - Phi Index Method
  - Infiltration Capacity curves (Horton Method)
- Unit Hydrographs
  - Convolution
  - NRCS (previously SCS) unit hydrograph
  - Others: Snyder, Clark
- Conceptual Models
  - Kinematic Wave
- All of them use some assumptions to substract the initial abstractions and other runoff losses

Base Flow Separation

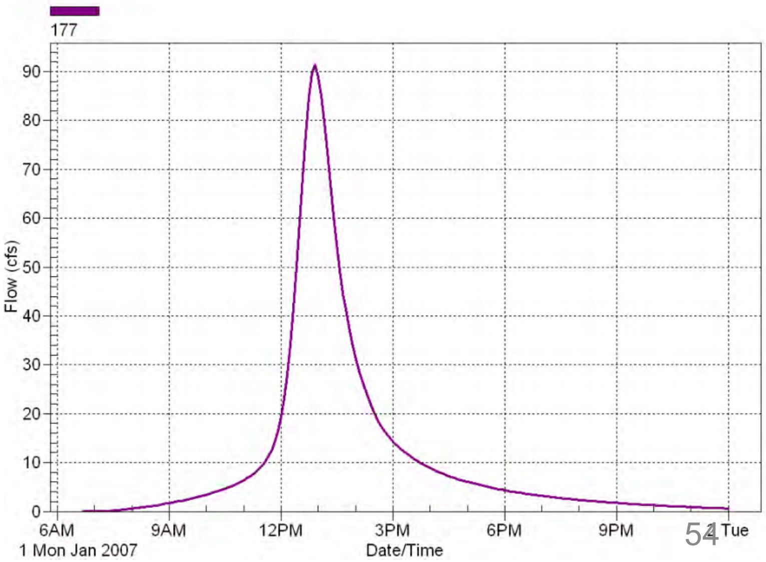
- Real hydrographs in perennial stream flow include a discharge in the stream previous to the storm generated discharge.
- This component is called BASE FLOW
- This contribution comes from the groundwater
- The base flow must be separated from the original storm hydrograph to obtain the DIRECT STORM RUNOFF

Base Flow Separation



STORM HYDROGRAPH
with BASE FLOW

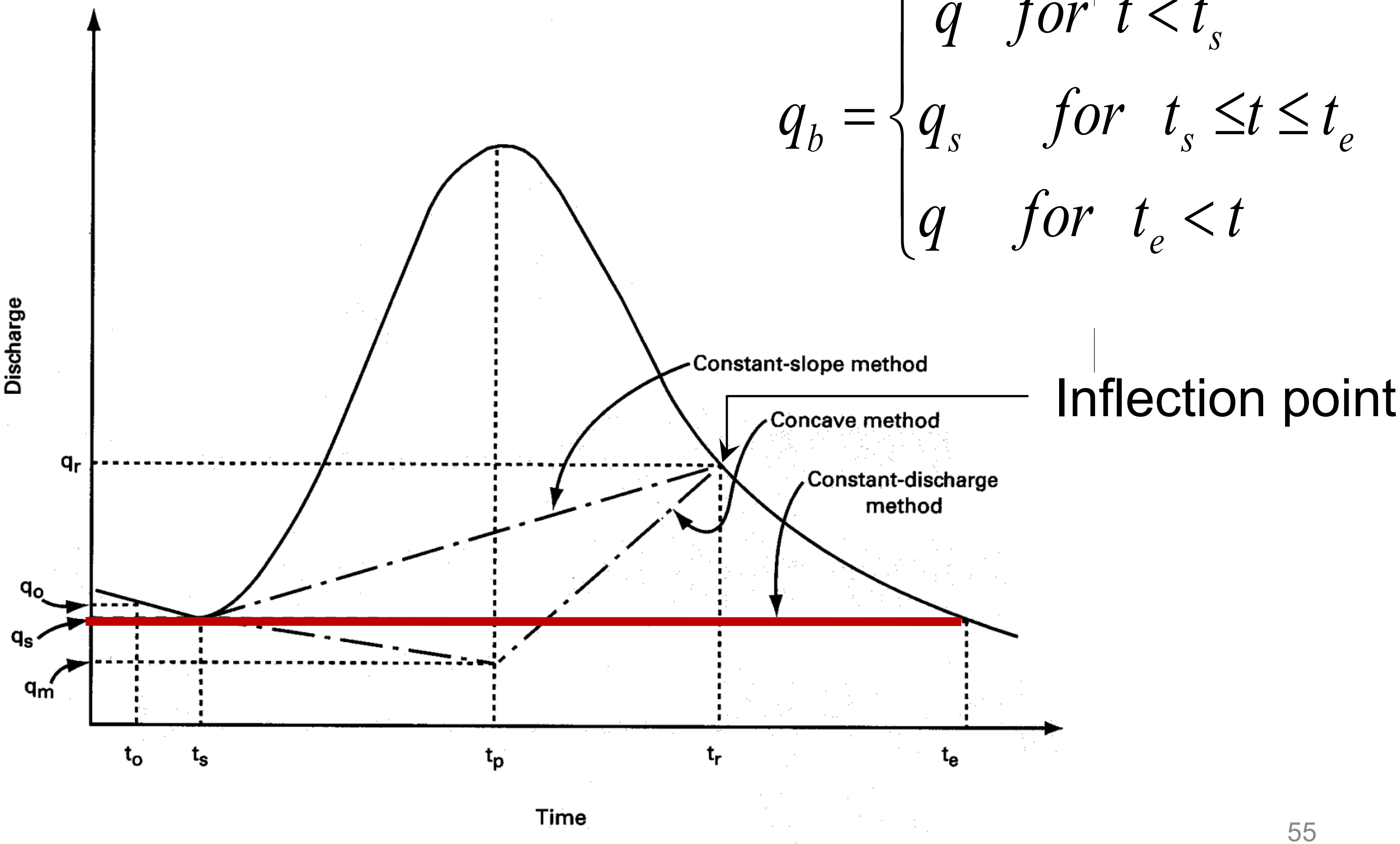
DIRECT STORM
RUNOFF



Base Flow separation methods

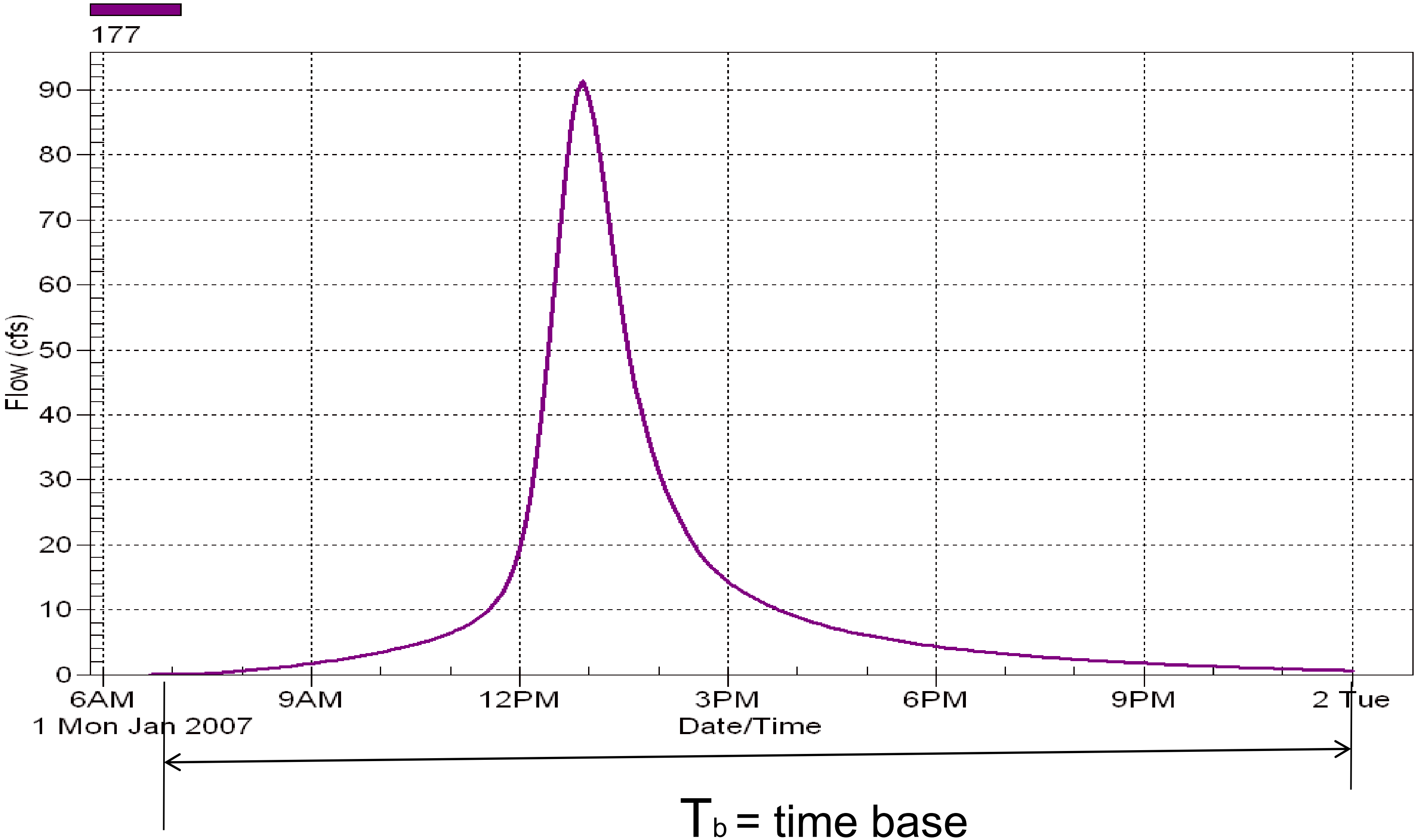
CONSTANT DISCHARGE METHOD

$$q_b = \begin{cases} q & for \ t < t_s \\ q_s & for \ t_s \le t \le t_e \\ q & for \ t_e < t \end{cases}$$



Baseflow separation methods.

Direct storm runoff hydrograph



T_b = time base

Unit Hydrograph

- A Unit Hydrograph of T hours (or minutes) is the hydrograph produced by a rainfall with a **unit direct runoff depth** over the drainage area (1 inch or 1 milimeter) as a result of a storm of T hours (or T minutes) of effective duration

Synthetic Unit Hydrographs

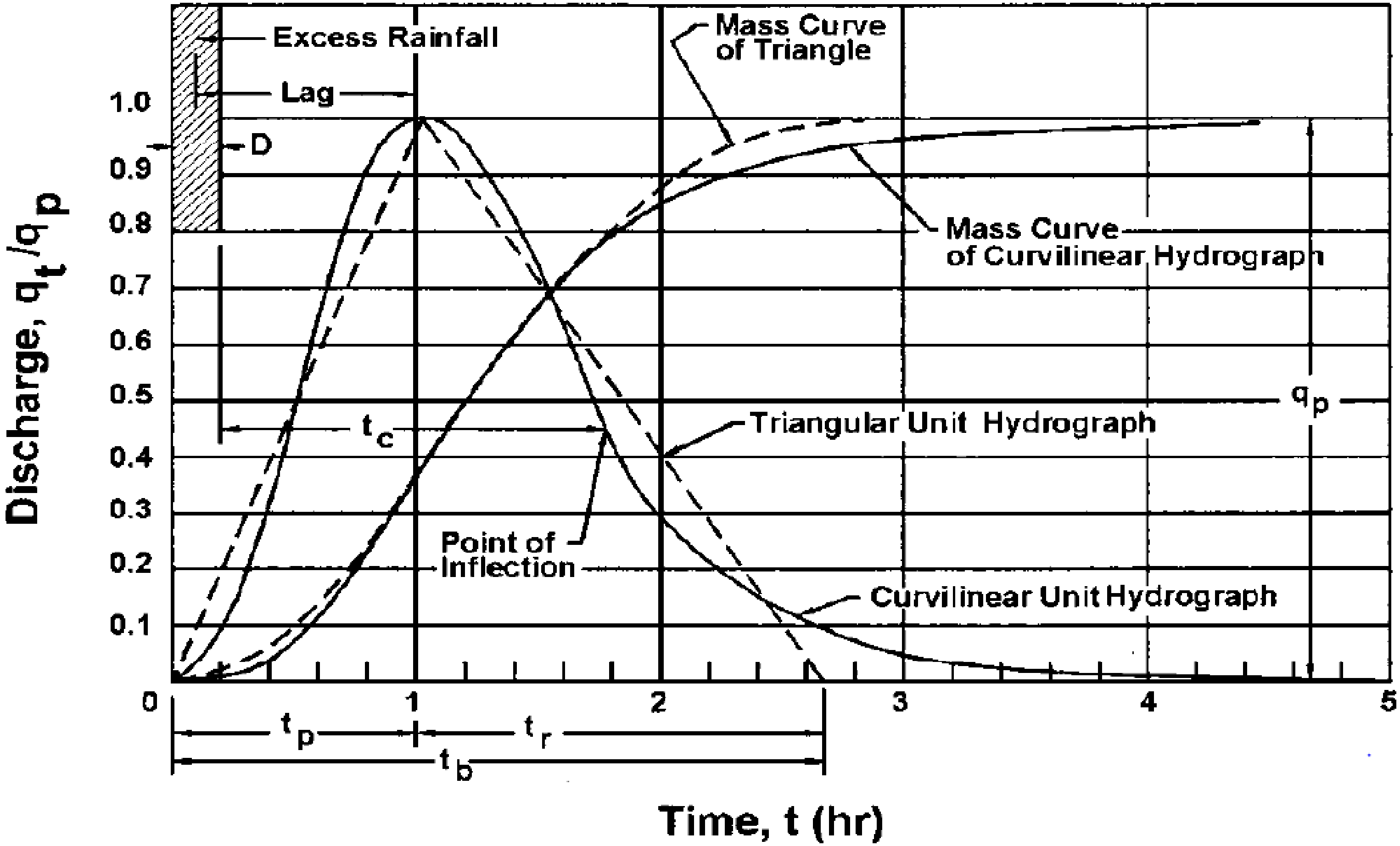
NRCS (SCS) Methods

- SCS developed a large number of UHs for gauged watersheds ranging in size and geographic location.
- These UHs were averaged to form one UH representative for all other watersheds.

Synthetic Unit Hydrographs

- SUH are used for ungauged watersheds where no historical records exist.
- The most commonly used SUH was developed by the NRCS
- Two such hydrographs exist
- Here only the triangular SUH is provided
- For a triangular hydrograph three points are required:
    1. Peak discharge
    2. Time to the peak discharge
    3. Total duration of the unit hydrograph

Dimensionless curvilinear and triangular NRCS synthetic unit hydrographs



The curvilinear hydrograph is used for design work !

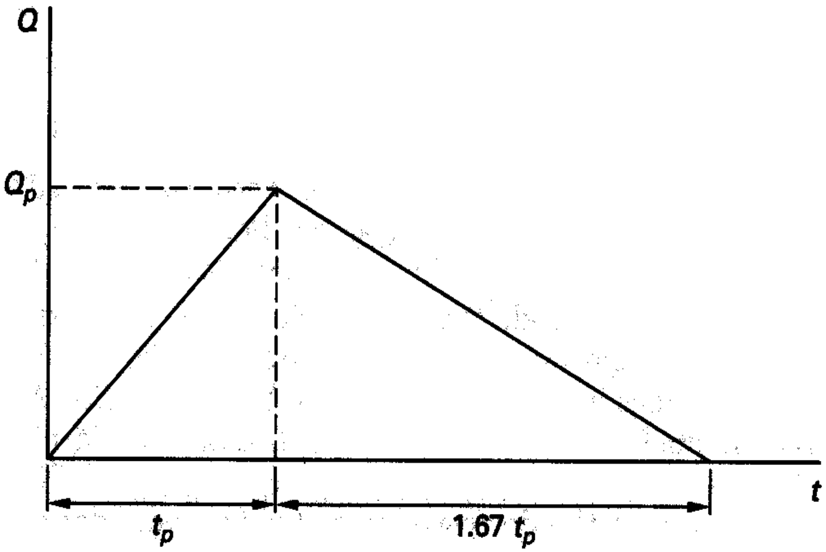
Synthetic Unit Hydrographs

NRCS synthetic unit triangular hydrograph

$$Q_p = \frac{0.756\,A_{acres}}{t_p} \qquad Q_p = \frac{484\,A_{millas\ 2}}{t_p}$$

$$t_p = 0.67 t_c \qquad t_b = 2.67 t_p$$

For the SCS-CN method the time of concentration is 1.67 times the lag time.
The constant 484 is mostly for developed areas.
Mountainous watersheds use 600
Flat-swampy areas use 300



Synthetic Unit Hydrographs

NRCS (SCS) Equations

- Peak Flow, cfs

$$Q_p = \frac{484A}{T_R}$$

- Time to Rise, hr

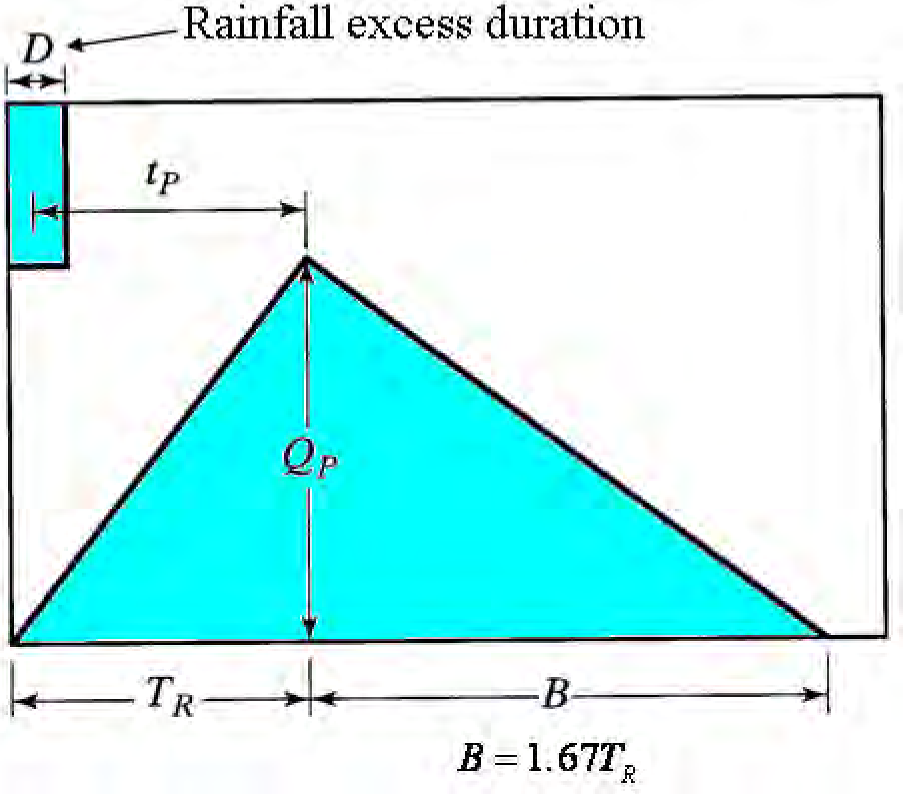
$$T_R = \frac{D}{2} + t_p$$

- Rainfall excess duration

$$D = 0.133t_c$$

- Time to peak flow

$$t_p = 0.6t_c$$



Example

- ***Given:*** *The following watershed conditions:*

Watershed is commercially developed

Watershed area = 0.463 mi^2

Time of concentration = 1.34 hr

Q_D = 1 in (UH)

- *Step 1: Calculate time to peak flow using*

$t_p = 2/3\ t_c = 0.89\ hr$

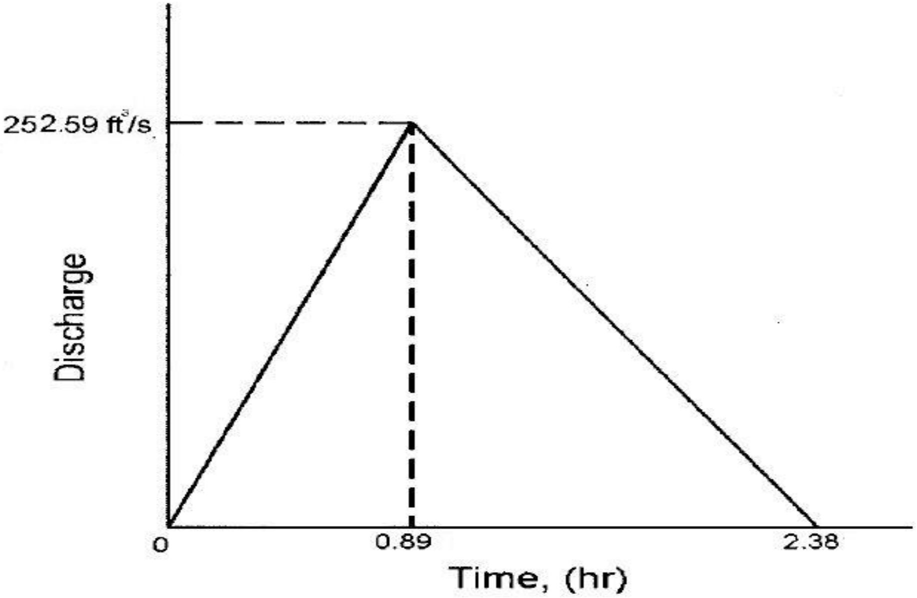
Step 2: Calculate peak flow

$$Q_p = \frac{484 A_{millas2}}{t_p} = 484\frac{0.463}{0.89} = 251.8\ cfs$$

Step 3: Calculate time base of UH.

$$t_b = 2.67 t_p = 2.38\ hr$$

Step 4: Draw resulting triangular UH



CN Tables

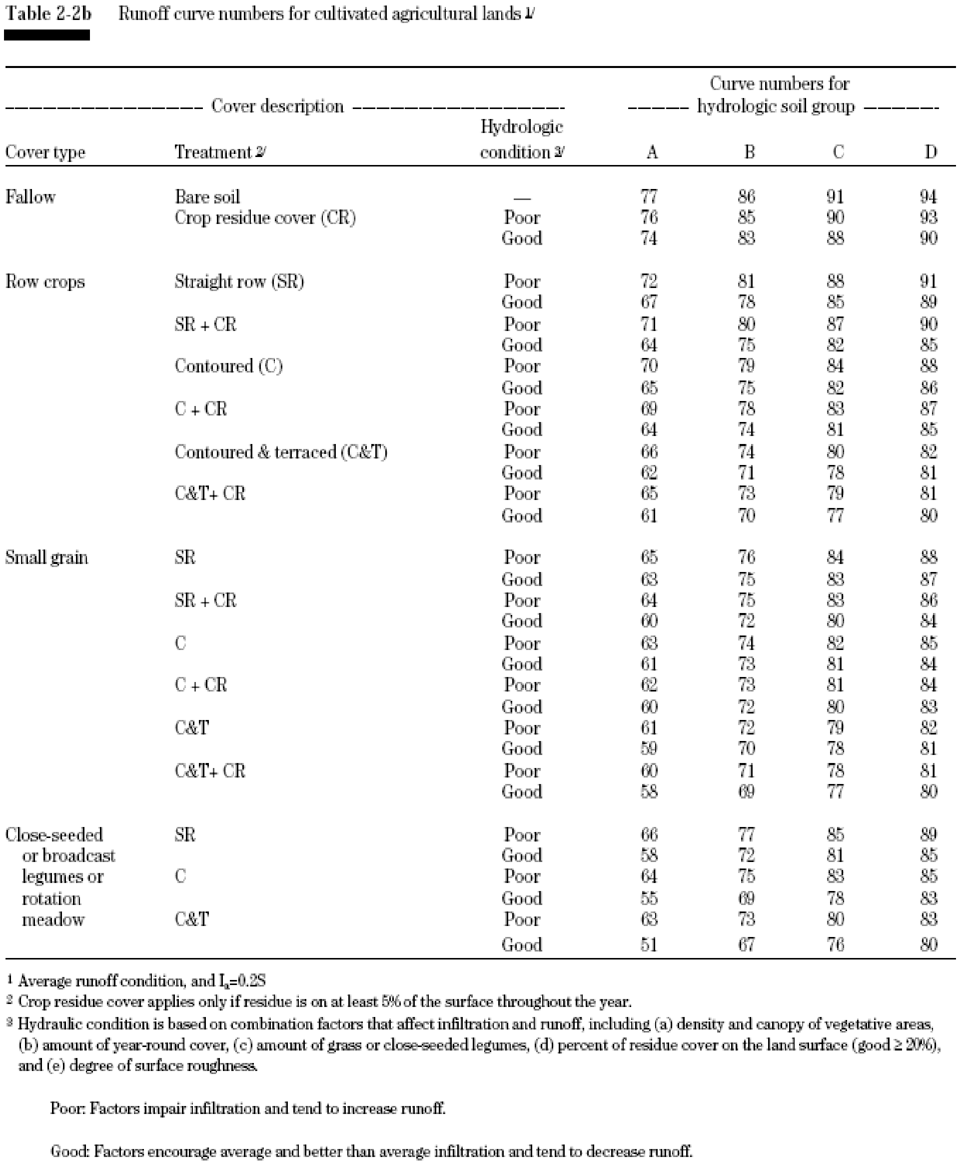


Table 2-2b Runoff curve numbers for cultivated agricultural lands [1]

Cover description			Curve numbers 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

[1] Average runoff condition, and I_a=0.2S

[2] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

[3] Hydraulic condition is based on combination 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 roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

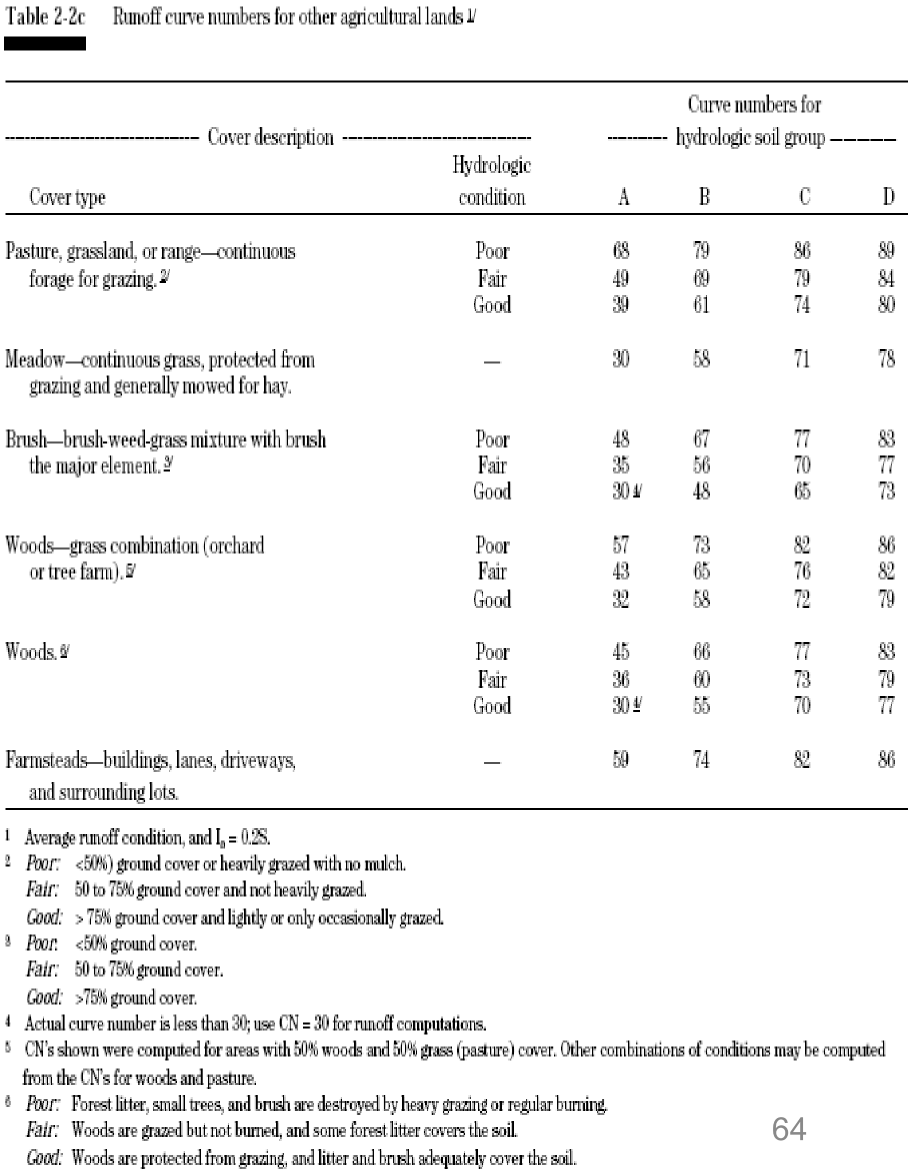


Table 2-2c Runoff curve numbers for other agricultural lands [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing. [2]	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.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. [3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 [4]	48	65	73
Woods—grass combination (orchard or tree farm). [5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. [6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 [4]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

[1] Average runoff condition, and I_a = 0.2S.

[2] *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.

[3] *Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

[4] Actual curve number is less than 30; use CN = 30 for runoff computations.

[5] CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

[6] *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.

CN Tables

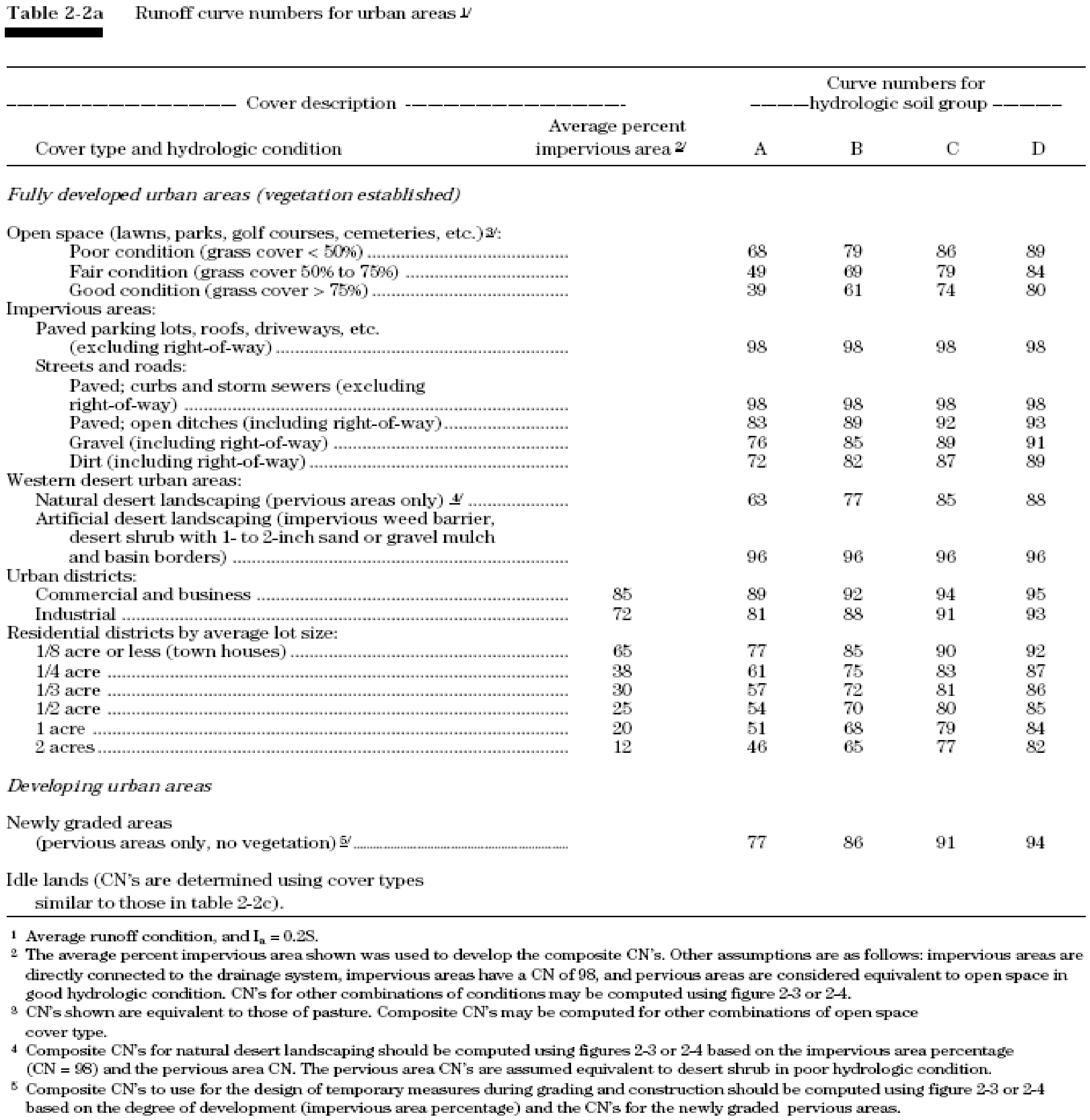


Table 2-2a Runoff curve numbers for urban areas [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

---

[1] Average runoff condition, and $I_a = 0.2S$.

[2] The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

[4] Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Module 3

Hydraulics: Open Channel Flow

OPEN CHANNEL FLOW

REVIEW

Walter F. Silva Araya, Ph.D., P.E

Flow Regimes

Flow Regimes	**Froude Number**
Subcritical	<1
Critical	=1
Supercritical	>1

$$F_r = \frac{V}{\sqrt{gD_h}}$$

V = Average flow velocity
g = Acceleration of gravity
D_h = Hydraulic depth

Uniform Flow

- Balance between shear and gravity forces
- Usually computed with **Manning's** equation

$$V = \frac{C}{n} R^{2/3} S^{1/2}$$

C=1 for SI

C=1.49 for U.S.

or

$$Q = \frac{CAR^{2/3} S^{1/2}}{n} = K\sqrt{S}$$

K=conveyance

S=bottom slope

n=Manning's coefficient

Energy Losses in Channels

- Friction: $\boxed{h_f = LS_f}$

S_f=slope of the energy line

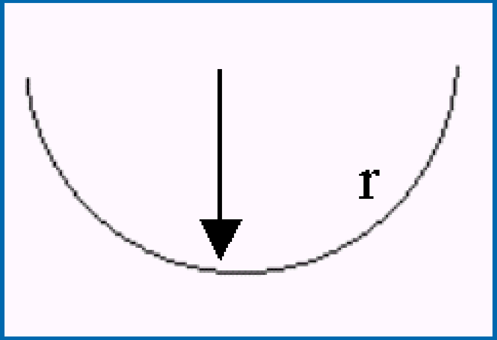
•If uses Manning's equation:

$$\boxed{h_f = \frac{Ln^2V^2}{R^{4/3}}}$$ S.I. or $$\boxed{h_f = \frac{Ln^2V^2}{2.208R^{4/3}}}$$ U.S.

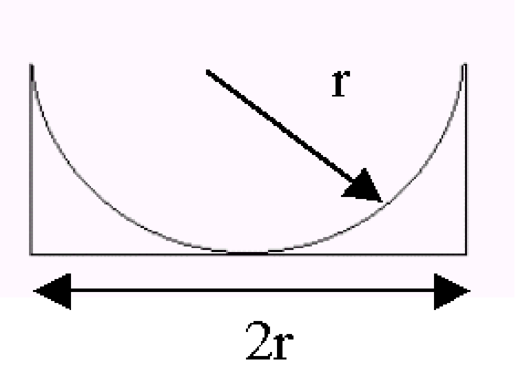
Most efficient Cross Section

- The most efficient open channel cross section will maximize the flow for a given Manning's coefficient, slope and flow area.
- This requires that the hydraulic radius be maximized. Therefore, for a given flow area the wetted perimeter will be minimum.
- This criteria apply only to rigid channels.

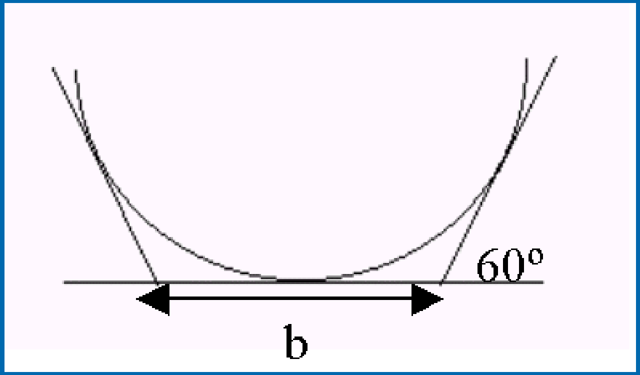
Most efficient rigid channels cross sections



circular

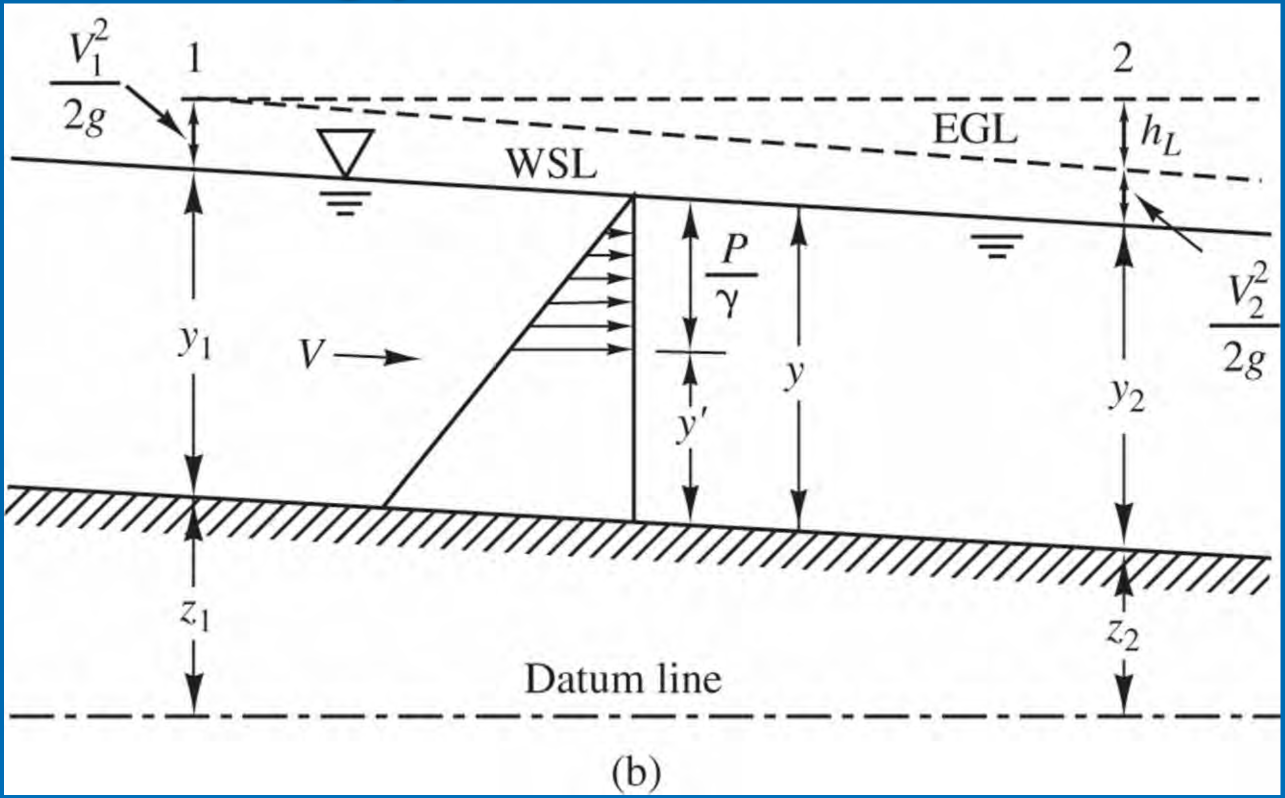


rectangular



trapezoidal

Energy in Open Channels



(b)

TOTAL ENERGY HEAD

$$H = Z + y + \frac{V^2}{2g}$$

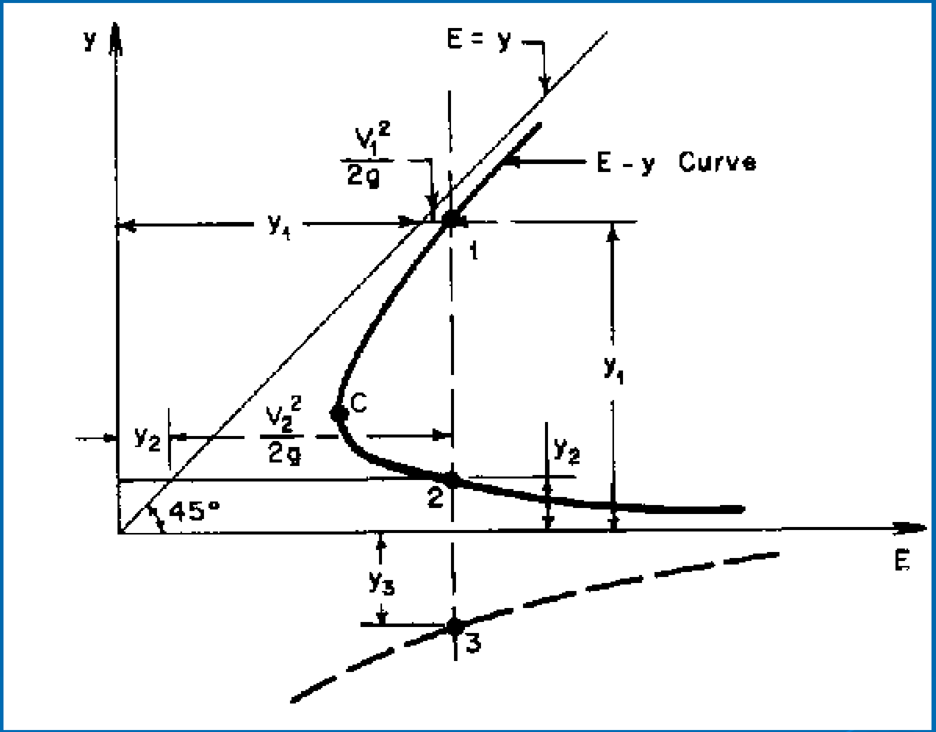
SPECIFIC ENERGY

$$E = y + \frac{V^2}{2g}$$

Specific Energy

$$E = y + \frac{V^2}{2g}$$

$$E = y + \frac{Q^2}{2gA^2}$$



y_1 and y_2 are alternate depths

$y_{1=}$supercritical flow, $F_r>1$

$y_{2=}$subcritical flow, $F_r<1$

y_c= critical depth, F_r =1 (minimum specific energy)

Specific Force:

- Is the total force per unit weight acting on the channel cross section.

$$F_s = \frac{Q^2}{Ag} + \bar{d}A$$

- $\bar{d}$ is the depth of the center of gravity of the cross section

Critical Flow and Critical Depth

- Critical Depth is the flow depth corresponding to the minimum specific energy. Corresponds to F_r =1

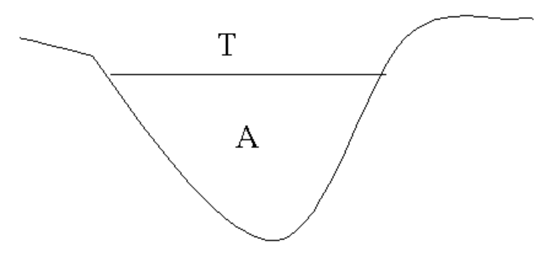
$$Fr = 1 = \frac{V}{\sqrt{gD}}$$

$$V = \sqrt{g(A/T)}$$

$$V^2 = \frac{gA}{T}$$

$$Q^2 = \frac{g}{AT}$$

$$Q = A\sqrt{gD}$$



For rectangular channel

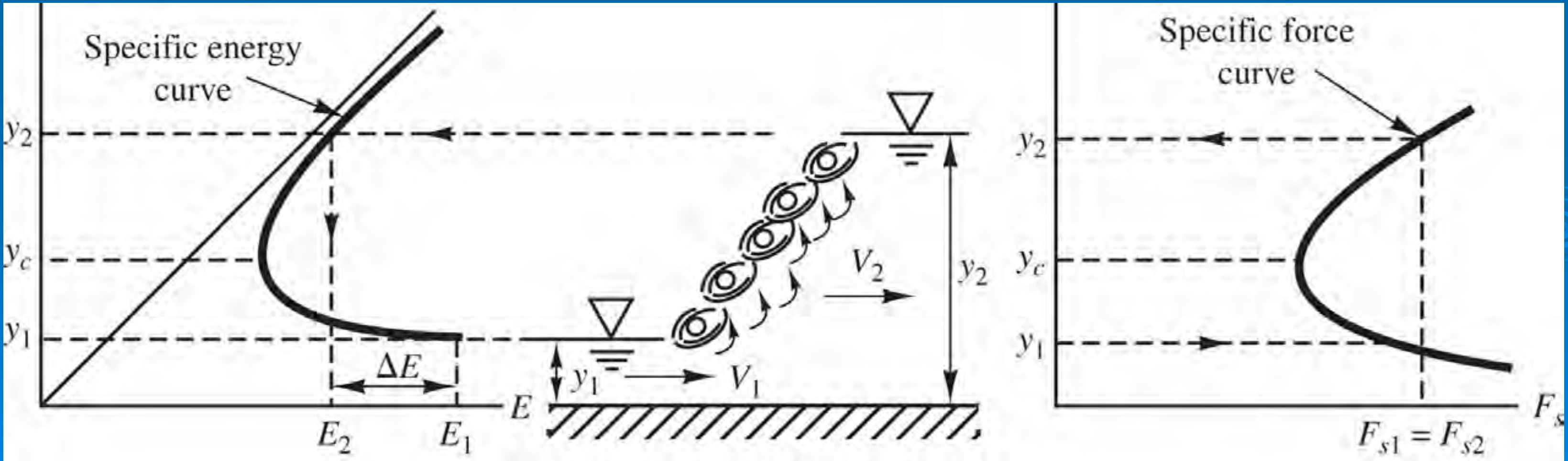
- D=y A=by

$$y_c{}^3 = \frac{Q^2}{gb^2}$$

$$y_c = \frac{2}{3}E_c$$

Hydraulic Jump

- Occurs when the flow changes abruptly from supercritical to subcritical.
- y_1 and y_2 are called CONJUGATE depth



Hydraulic Jump (cont.)

- For a rectangular channel:

$$\frac{y_2}{y_1} = \frac{1}{2}(\sqrt{1+8F_{r1}^{\ 2}} - 1)$$

$$\frac{y_1}{y_2} = \frac{1}{2}(\sqrt{1+8F_{r2}^{\ 2}} - 1)$$

If y_1 and y_2 are known, then the upstream velocity is:

$$V_1^{\ 2} = \left(\frac{gy_2}{2y_1}\right)(y_1 + y_2)$$

The energy loss at the jump is:

$$\Delta E = \frac{(y_2 - y_1)^3}{4y_1y_2}$$

OPEN CHANNEL FLOW

1) Consider a rectangular channel 3 m wide laid on a 1^0 slope. The channel is built of rubble cement (Manning's n = 0.020), what is the uniform flow rate when the water depth is 2 m?

Find the parameters:

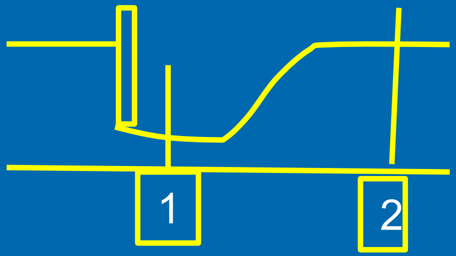
A = 2x3 = 6 m2
P = 3+2x2 = 7 m
R = 6/7 = 0.857
S = tan (1^o) = 0.0175

Apply Manning's equation directly with $k_M = 1$:

$$Q = \frac{6 \times (0.857)^{2/3} \times (0.0175)^{1/2}}{0.02} = 35.76 \ m^3/s$$

OPEN CHANNEL FLOW

The Figure shows a sluice gate in a 35-ft wide rectangular channel. At section 1, Y_1 = 3.8 ft and the velocity is 50 ft/s. What is the Froude number at section 2 ?



Q = V_1 A = 50 x 3.8 x 35 = 6,650 cfs

$$F_1 = \frac{V_1}{\sqrt{gD_1}} = \frac{50}{\sqrt{(32.2)(3.8)}} = 4.52$$

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right) = 5.91$$

y_2 = 5.91 y_1 = 5.91 (3.8) = 22.46 ft

V_2 = Q_2/A_2 = 6,650/(35)(22.46) = 8.46 ft/s

$$F_2 = \frac{V_2}{\sqrt{gD_2}} = \frac{8.46}{\sqrt{(32.2)(22.46)}} = 0.31$$

From NCEES Review

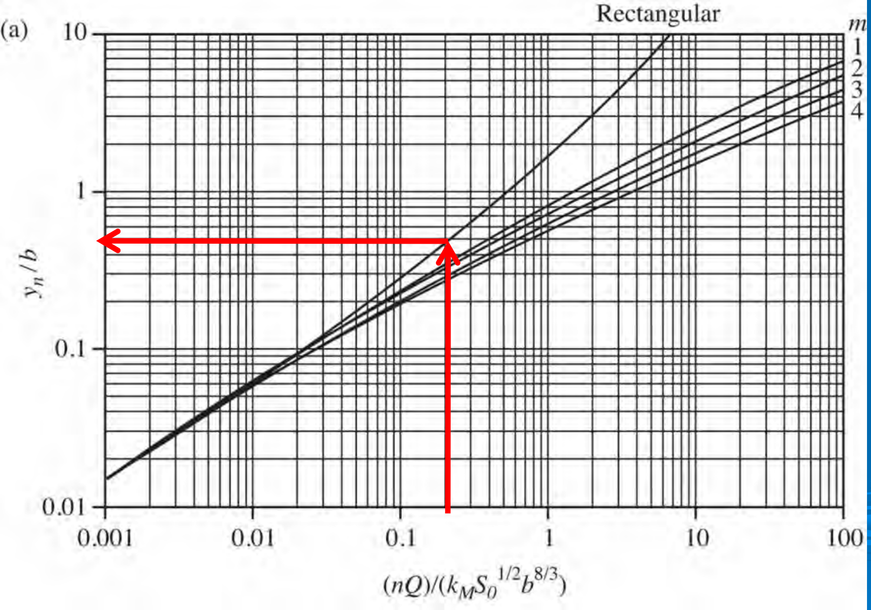
OPEN CHANNEL FLOW

2) For the same previous channel, if Manning's roughness factor is n = 0.020 and Q = 24 m^3/s, what is the normal depth?

Get parameter for design chart:

$$\frac{Qn}{S^{1/2} b^{8/3}} = \frac{A R^{2/3}}{b^{8/3}}$$

$$\frac{24 \times 0.02}{(0.0175)^{1/2} (3)^{2.667}} = 0.194$$



Use design graph with 0.194 to get:

$y_n/b = 0.5$ (b = channel bottom width)

Obtain $y_n = 1.5$ m

OPEN CHANNEL FLOW

3) What is the flow velocity in a 0.5 m diameter sewer pipe with a discharge of 0.07 m^3/s if the slope is 0.001 and the pipe material is precast concrete? Consider that Manning's n varies with depth.

a) Apply Manning's equation to get the flow velocity for a full pipe:

$$\begin{aligned} v_{\text{full}} &= \left(\frac{1.00}{n}\right) R^{\frac{2}{3}} \sqrt{S} \\ &= \left(\frac{1.00}{0.015}\right) (0.125 \text{ m})^{\frac{2}{3}} \sqrt{0.001} \\ &= 0.53 \text{ m/s} \end{aligned}$$

b) The pipe discharge for full pipe conditions is:

$$\begin{aligned} Q_{\text{full}} &= v_{\text{full}} A \\ &= \left(0.53 \ \frac{\text{m}}{\text{s}}\right) \left(\frac{\pi}{4}\right) (0.5 \text{ m})^2 \\ &= 0.10 \text{ m}^3/\text{s} \end{aligned}$$

c) Therefore the ratio Q/Q_{full} is:

$$\frac{Q}{Q_{\text{full}}} = \frac{0.07 \ \frac{\text{m}^3}{\text{s}}}{0.10 \ \frac{\text{m}^3}{\text{s}}} = 0.7$$

OPEN CHANNEL FLOW

d) With the value Q/Qfull = 0.7 go to the graph for normal depth in circular pipes to get the ratio d/D:

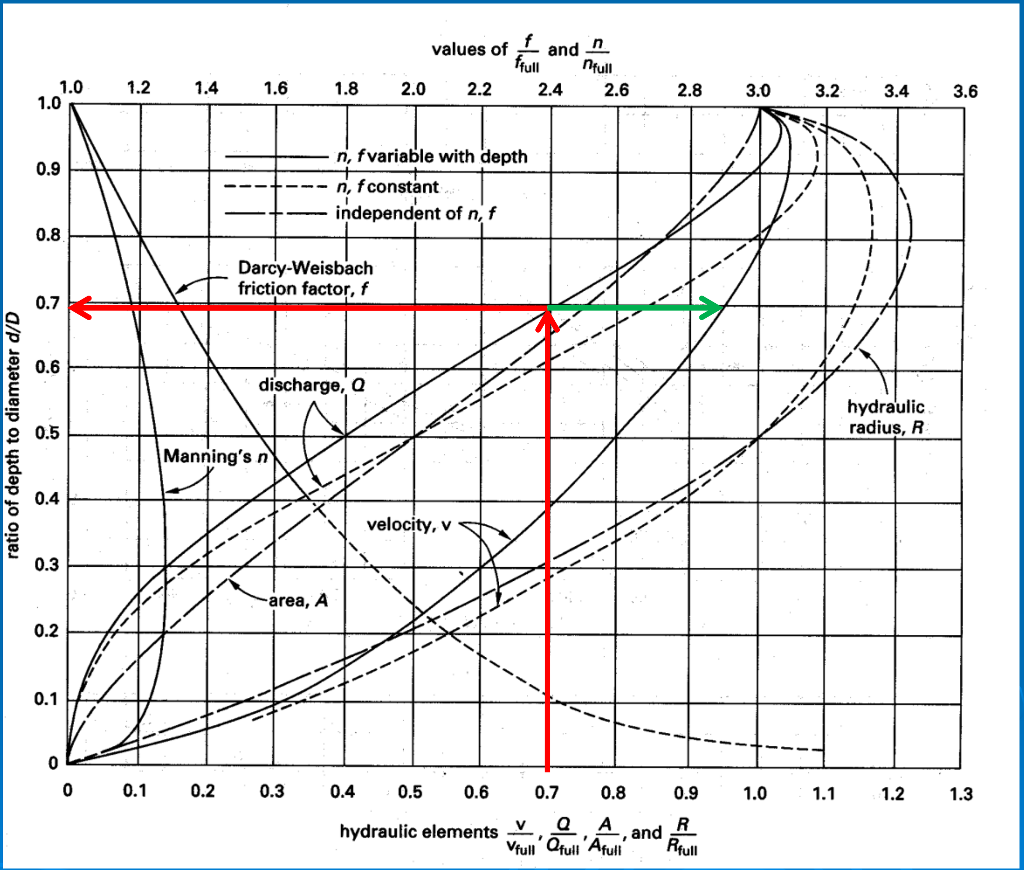
$$d/D = 0.68$$

$$\mathrm{v}/\mathrm{v}_{\mathrm{full}} = 0.94$$

e) Get the flow depth and flow velocity:

$$\mathrm{v} = (0.94)\left(0.53\ \frac{\mathrm{m}}{\mathrm{s}}\right) = 0.50\ \mathrm{m/s}$$

$$d = (0.68)(0.5\ \mathrm{m}) = 0.34\ \mathrm{m}$$

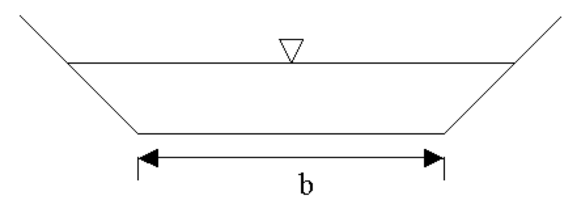


OPEN CHANNEL FLOW

4) A trapezoidal channel with a bottom width of 4 m and side slopes of $z = 1.5$ is carrying a discharge of 50 m^3/s at a depth of 3 m. Determine the following:

a) The alternate depth for the same specific energy
b) The critical depth
c) The uniform flow depth for a slope of 0.0004 and n=0.022

SOLUTION: Alternate dep



a) The specific energy is given by:

$$E = y + \frac{Q^2}{2gA^2}$$

Compute the area:

$$A = (b + my)y = [4\ \text{m} + 1.5(3\ \text{m})](3\ \text{m}) = 25.5\ \text{m}^2$$

Compute the specific energy for the given depth

$$E = Q^2/(2gA^2) + y = 50^2/[2\cdot 9.81(25.5)^2]+3\text{m} = 3.20\ \text{m}$$

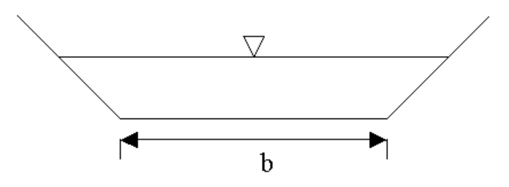
Compute the alternate depth solving the specific energy equation by trial and error:

$$3.2\text{m} = 50^2/(2g\{(4 + 1.5y)y\}^2) + y \Longrightarrow \mathbf{y = 1.38\ m}$$

OPEN CHANNEL FLOW

5) A trapezoidal channel with a bottom width of 4 m and side slopes of z = 1.5 is carrying a discharge of 50 m^3/s at a depth of 3 m. Determine the following:

a) The alternate depth for the same specific energy
b) The critical depth
c) The uniform flow depth for a slope of 0.0004 and n=0.022



SOLUTION: Critical depth

b) Use the graphical procedure computing:

$$(Qm^{3/2})/(g^{1/2}b^{5/2})$$

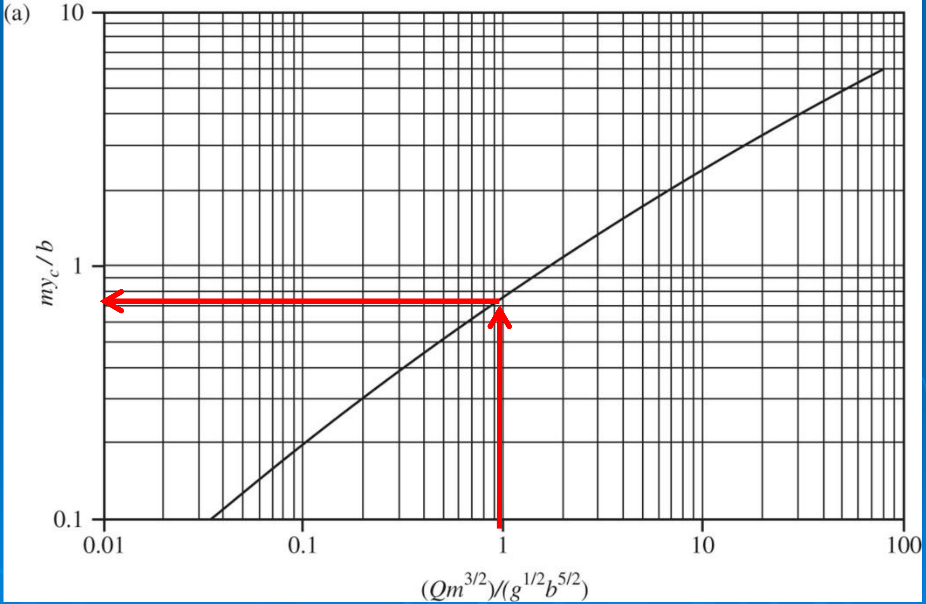
$50x1.5^{1.5}/(9.81^{0.5}x4^{2.5} = 0.916$

Go to design graph:

$my_c/b = 0.71$

Therefore:

y_c= 0.71x4/1.5 = 1.9m



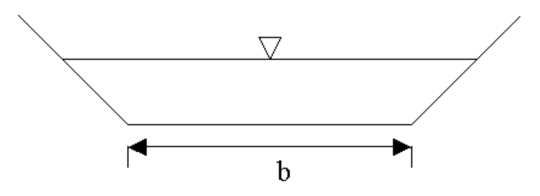
OPEN CHANNEL FLOW

6) A trapezoidal channel with a bottom width of 4 m and side slopes of z = 1.5 is carrying a discharge of 50 m^3/s at a depth of 3 m. Determine the following:

a) The alternate depth for the same specific energy

b) The critical depth

c) The uniform flow depth for a slope of 0.0004 and n=0.022



SOLUTION: Normal depth

c) Use the graphical procedure computing: $(nQ)/(k_M S_0^{1/2} b^{8/3})$

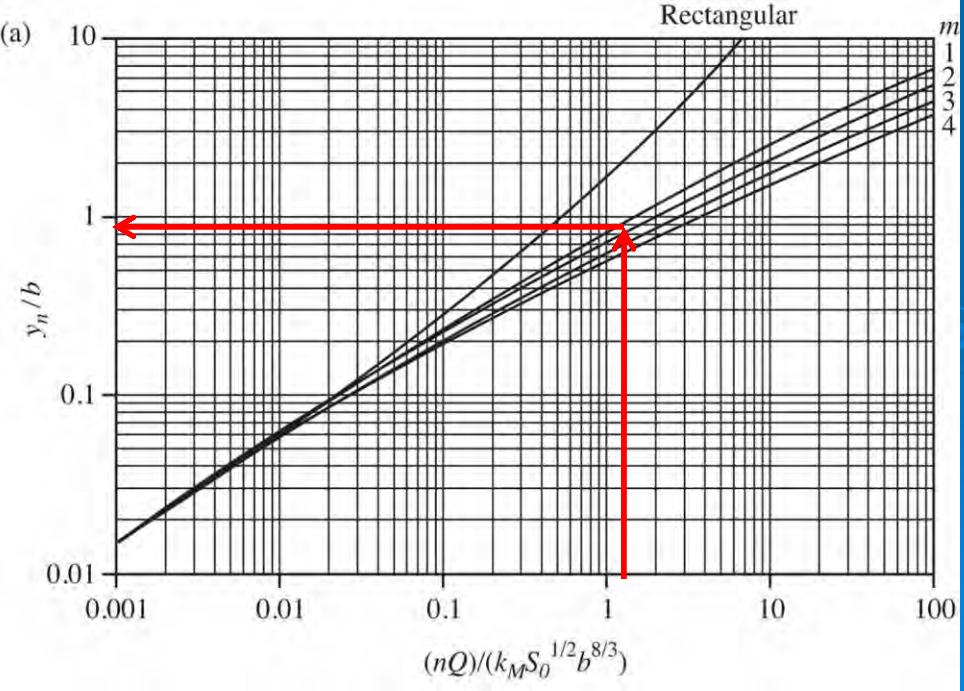
$0.022x50/(0.0004^{0.5}x4^{2.67})= 1.364$

Go to design graph and get:

$y_n/b = 0.90$

Therefore:

y_n= 0.90x4 = 3.60 m



OPEN CHANNEL FLOW

7) How much it the width of a rectangular channel with n = 0.01, that carries 500 cfs at a depth of 6 ft on a slope of 0.0004 ?

Use Manning's formula with: Q = 500 cfs, n = 0.01

$A = 6w$ and $R = 6w/(w + 12)$

$Q = 1.49 A R^{0.66} S^{0.5} /n$

$Q = 1.49 (6w) \{6w/(w+12)\} (0.00004)^{0.5} / n$

Solve by trial and error to get w = 13.1

(this method requires iterations!)

OPEN CHANNEL FLOW

8) The width of a rectangular channel with n = 0.01, that carries 500 cfs at a depth of 6 ft on a slope of 0.0004 is ?

Use discharge factors:

Compute $K = Q\,n/(y^{8/3}\,S^{0.5})$

n = 0.01, S = 0.0004, Q = 500 cfs, y = 6 ft

$K = 2.11$

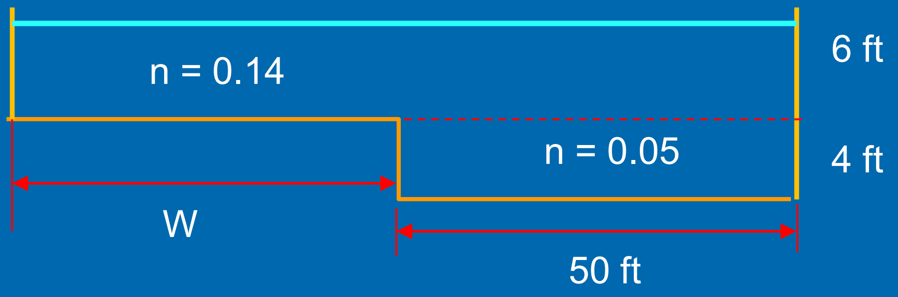
Go to table for values of K. K = 2.11 lies between 2.22 and 2.09. By interpolation y/b = 0.458

Then b = 6 / 0.458 = 13.1 ft

(this method is direct!)

OPEN CHANNEL FLOW

9) A composed channel has a longitudinal slope of 0.001 ft/ft. The design flow is 3,000 cfs. What should be the minimum width (W) of the overbank section to keep the water depth in the channel no greater than 10 ft ?



A_1= 50 x 10 = 500 ft2
P_1 = 50 + 2(4) + 6 = 64 ft
$R_1 = A_1/P_1$ = 500/64 = 7.81 ft
$K_1 = (1.49/n1)(A1)(R1)^{2/3}$
$K_1 = (1.49/0.05)\ (500)\ (7.81)^{2/3}$ = 58,508 cfs
A_2 = 6W
P_2 = W + 6
R_2 = 6W/(W+6)
$K_2 = (1.49/n_2)\ (A_2)(R_2)^{2/3} = (1.49/0.14)(6W)(W+6)^{2/3}$

$Q = (K_1 + K_2)\ S^{1/2} = 3000$ (K is the Conveyance)

$3000 = \{58{,}508 + [(1.49/0.14)\ (6W)(W+6)^{2/3}]\}\ (0.001)^{1/2}$
Simplifying:
$(6W)^{5/3}/(W+6)^{2/3} = 3{,}416.4$

Solving by trial and error:
W = 177 ft

From NCEES PE Review

Hydraulic Jump

15) A 10 ft rectangular channel carries 500 cfs of water at a 2 ft depth before entering a jump. Compute the downstream water depth and the critical depth

SOLUTION

The equation for the sequent depth is:

$$\frac{y_2}{y_1} = \frac{1}{2}(\sqrt{1+8F_{r1}^{\ 2}} - 1)$$

Need to compute the Froude number upstream of the jump, F_{r1}

The discharge per unit width in a rectangular channel is: Q/b

$$q = \frac{500}{10} = 50 \text{ ft}^3/\text{sec} \cdot \text{ft}$$

The critical depth in a rectangular channel is given by:

$$y_c^{\ 3} = \frac{Q^2}{gb^2}$$

Using q = Q/b results in:

$$y_c = \sqrt[3]{\frac{50^2}{32.2}} = 4.27 \text{ ft}$$

Compute the Froude number for y_1 = 2.0 ft as:

$$= \frac{V_1}{\sqrt{gy_1}} = 3.12$$

Substituting in the sequent depths formula and solving for y_2:

$$\frac{y_2}{2.0} = \frac{1}{2}\left(\sqrt{1 + 8(3.12)^2} - 1\right)$$

→

$$y_2 = 7.88 \text{ ft}$$

Gradually Varied Flow

- Depth varies along the channel at a small rate

Assumptions:

- Small channel slope
- Hydrostatic pressure distribution
- The friction losses can be estimated using Manning’s equation

GVF Computation Methods

- Standard Step Method
  - Useful when the channel cross sections are known at specific sites
  - Is preferable for natural channels and rivewrs
  - Solves the energy equation iteratively
- Direct Step Method
  - Useful to estimate the distance between two know depths
  - Appropriate for prismatic channels such as culverts

Classification of Flow Profiles

Channel Slope

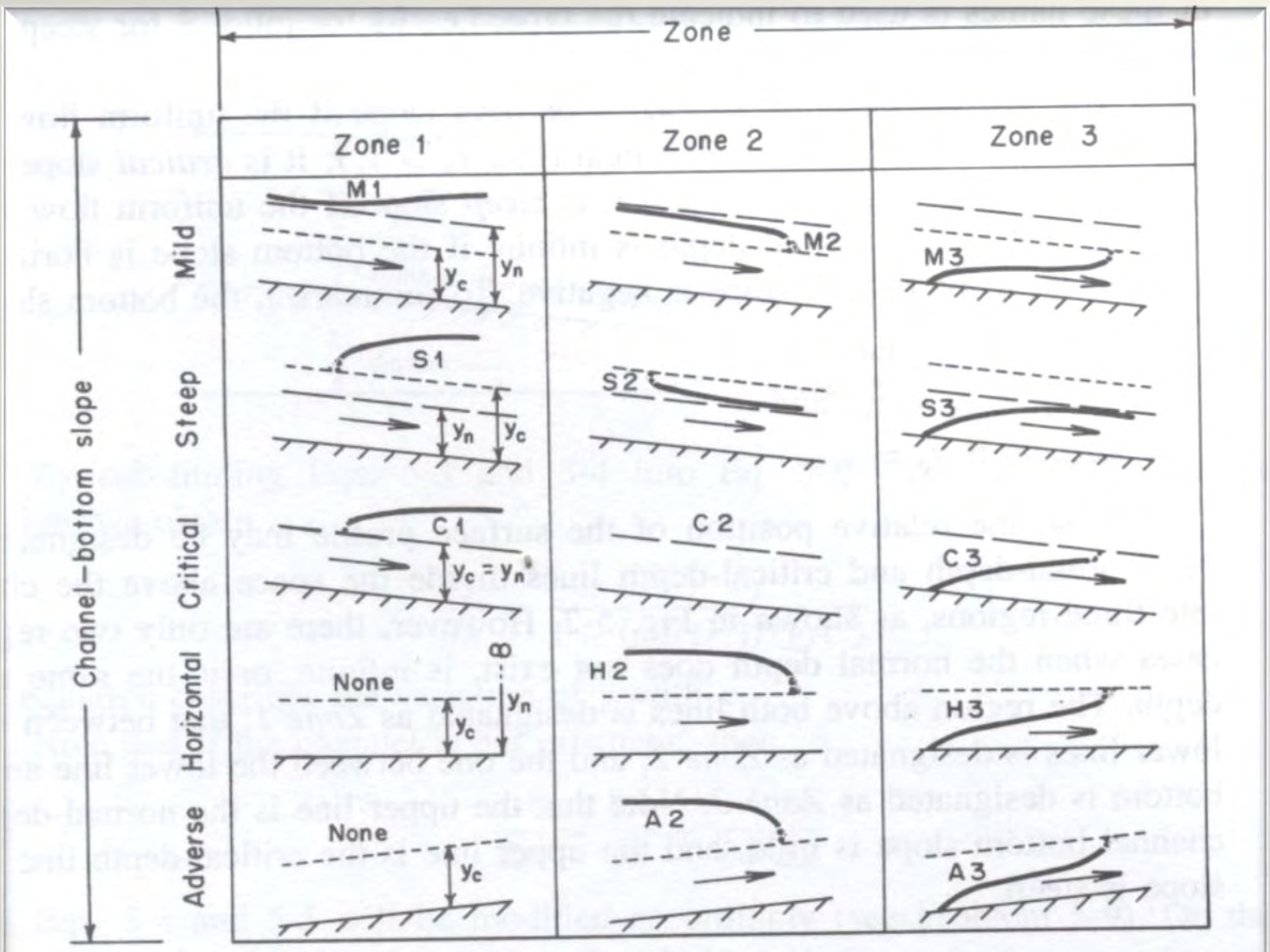
- Mild Channel:
  - Normal Depth > Critical Depth
  - Uniform flow is subcritical
  - Water surface profile type M
- Steep Channel:
  - Normal Depth < Critical Depth
  - Uniform flow is supercritical
  - Water surface profile type S
- Channel with Critical Normal Depth
  - Normal = Critical Depth
  - Water surface profile type C

Classification of Flow Profiles

Localization of the water surface

- Zone 1
  - Water surface elevation is above the normal and the critical depth
- Zone 2
  - Water surface elevation is between the normal and the critical depth
- Zone 3
  - Water surface elevation is below the normal and the critical depth

Classification of Water Surface Profiles



Water surface profiles.

Gradually varied Flow

16) A six foot diameter storm drain along a street lies on a uniform slope of 0.002. This storm drain discharges into a creek. During a recent event a peak discharge of 85 ft^3/s was measured when the depths between the beginning and end of the manholes were:

Upstream M.H. water El. = 4.3 ft

Downstream M.H. water El. = 3.5 ft

The pipe material is rough concrete.

What is the distance between these manholes?

Gradually Varied Flow Equation in terms of the specific energy:

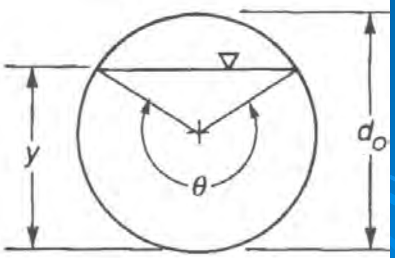
$$\Delta x = \frac{E_2 - E_1}{S_0 - S}$$

Obtain the properties of the circular pipe using equations for geometric properties:

$$\theta = 2\,cos^{-1}[1 - 2(y/d_0)]$$

Remember to use radians!!

$$A = \frac{1}{8}(\theta - \sin\theta)d_o^{\,2}$$



$$P = \frac{1}{2}\theta d_o$$

$$\theta = \pi + 2\sin^{-1}\left(\frac{2y}{d_o} - 1\right)$$

Gradually varied Flow

16) A six foot diameter storm drain along a street lies on a uniform slope of 0.002. This storm drain discharges into a creek. During a recent event a peak discharge of 85 ft^3/s was measured when the depths between the beginning and end of the manholes were:

Upstream M.H. water El. = 4.3 ft Downstream M.H. water El. = 3.5 ft

The pipe material is rough concrete.

What is the distance between the manholes?

SOLUTION

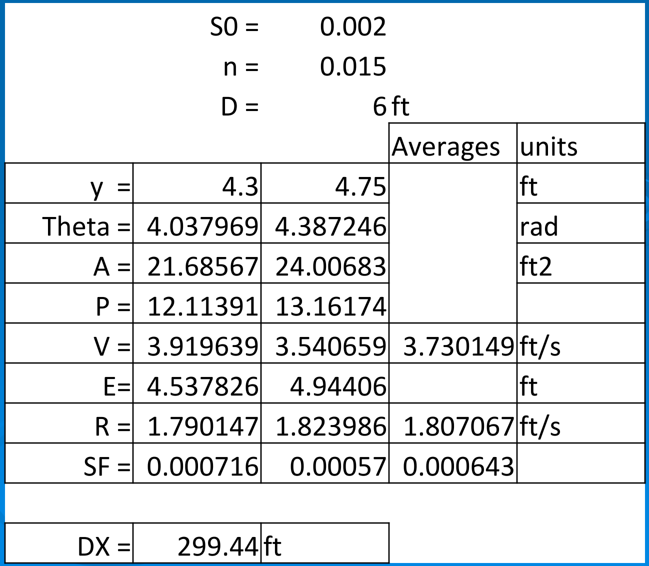
Use Manning n = 0.015. Assume constant discharge with respect to time and spa

Use the steady-state gradually varied flow equation:

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

Obtain the properties of the circular pipe

Using equations for geometric properties

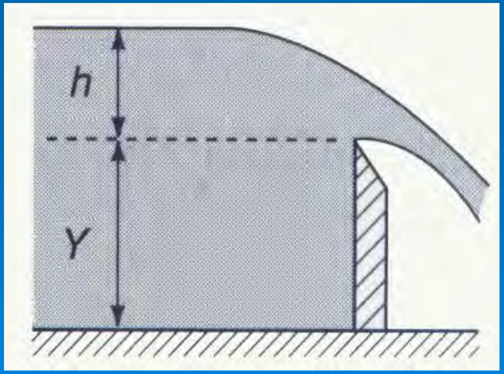


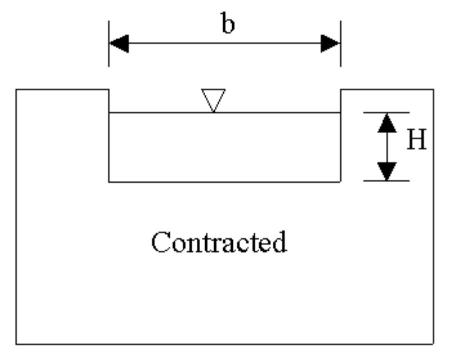
	S0 =	0.002		
	n =	0.015		
	D =	6	ft	
			Averages	units
y =	4.3	4.75		ft
Theta =	4.037969	4.387246		rad
A =	21.68567	24.00683		ft2
P =	12.11391	13.16174		
V =	3.919639	3.540659	3.730149	ft/s
E=	4.537826	4.94406		ft
R =	1.790147	1.823986	1.807067	ft/s
SF =	0.000716	0.00057	0.000643	
DX =	299.44	ft		

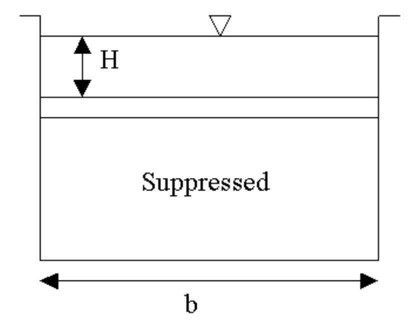
Weirs

- A weir is an obstruction in an open channel used for discharge measurement.
- **Sharp Crested Weirs**

$$Q = \frac{2}{3} C_d b \sqrt{2g} H^{3/2}$$







Discharge Coefficient

- There are equations in the literature to evaluate C_d
- Rehbock Equation

$$C_d = (0.6035 + 0.0813(H/y) + \frac{0.000295}{y})*(1+\frac{0.00361}{H})^{3/2}$$ [U.S. only]

$$C_d \approx (0.602 + 0.083(H/y))$$ [U.S. and S.I.]

When $H/y < 5$ ------ $0.61 < C_d < 0.62$

Summarizing the constants in the equation for the weir and using 0.61 we have:

$$Q = 1.84bH^{2/3}$$ [SI]

$$Q = 3.33bH^{2/3}$$ [U.S.]

$$Q = \frac{2}{3}C_d b\sqrt{2g}H^{3/2}$$

Weirs

- If the weir is not suppressed then the following correction should be applied

$$\boxed{b_{eff} = b_{actual} - 0.1NH}$$

Where N=1 for contraction on one side only

N=2 for contraction on both sides

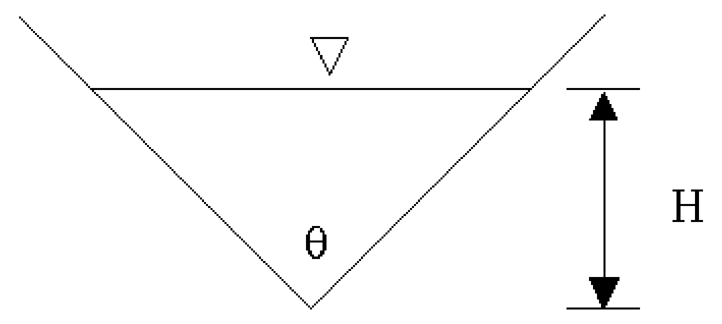
Triangular weirs

- Also called V-notch weirs
- Used to measured small flows
- Discharge coefficient depends on the angle θ
- $0.58<C_d<0.61$
- For $\theta=90^\circ$ $C_d=0.593$

$$Q = C_d (8/15)\tan(\theta/2)\sqrt{2g}H^{5/2}$$

$$Q=1.4H^{2.5} \qquad SI$$

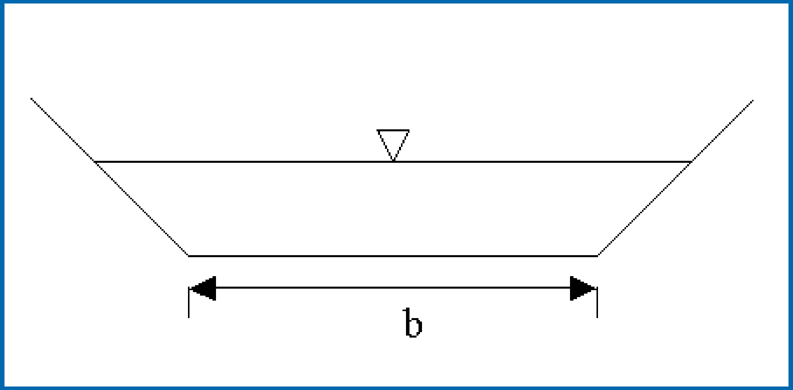
$$Q=2.5H^{2.5} \qquad US$$



Trapezoidal Weirs

$$Q = \frac{2}{3} C_d b \sqrt{2g} H^{3/2}$$

$C_d \approx 0.63$



For Cipoletti weirs:

$$Q = 1.86 b H^{3/2}$$ (SI)

$$Q = 3.367 b H^{3/2}$$ (US)

Broad Crested Weirs and Spillways

- A weir is broad-crested if the weir thickness is greater than half of the head, H.
- A spillway is a weir designed for a dam and have a cross section (known as ogee) which approximates the nappe from a sharp-crested weir.
- This cross section minimizes cavitation on the spillway.

Broad Crested Weirs and Spillways

- The discharge may be computed as:

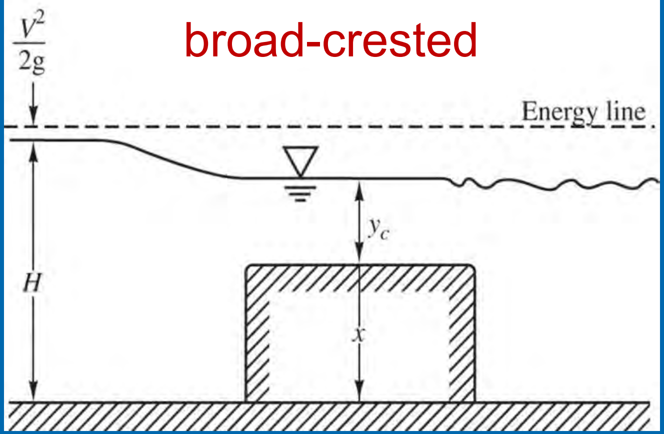
$$Q = \frac{2}{3} C_d b \sqrt{2g} H^{3/2}$$

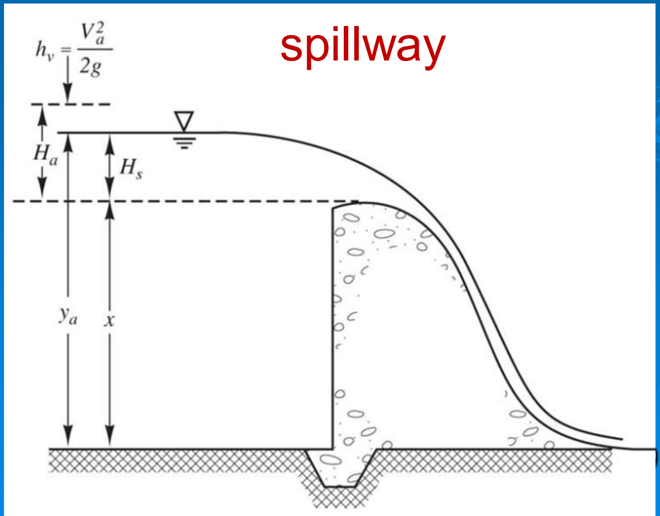
Broad –crested weirs

$0.5 < C_d < 0.57$

Ogee Spillways

$0.6 < C_d < 0.75$





Open Channels: Weirs

10) The crest of a sharp-crested, rectangular weir with two contractions is 2.5 ft high above the channel bottom. The crest is 4 ft long. A 4 in head exists over the weir. What is the velocity of approach in the channel?

SOLUTION

a) Draw a sketch:

b)Convert the height to feet and get the effective width using the correction due to contractions: The weir has two contractions therefore N = 2.

$$H = \frac{4\ \text{in}}{12\ \frac{\text{in}}{\text{ft}}} = 0.333\ \text{ft}$$

$$b_{\text{effective}} = b_{\text{actual}} - 0.1NH = 4\ \text{ft} - (0.1)(2)(0.333\ \text{ft}) = 3.93\ \text{ft}$$

c) Use Rehbock equation to get the discharge coefficient:

$$C_1 = \left(0.6035 + 0.0813\left(\frac{H}{Y}\right) + \left(\frac{0.000295}{Y}\right)\right)\left(1 + \frac{0.00361}{H}\right)^{\frac{3}{2}}$$

$$= \left(0.6035 + (0.0813)\left(\frac{0.333\ \text{ft}}{2.5\ \text{ft}}\right) + \frac{0.000295}{2.5\ \text{ft}}\right) \times \left(1 + \frac{0.00361}{0.333\ \text{ft}}\right)^{\frac{3}{2}} = 0.624$$

Open Channels: Weirs

SOLUTION

d) Obtain the discharge using the weir equation:

$$Q = \tfrac{2}{3} C_1 b \sqrt{2g} H^{\frac{3}{2}} = \left(\tfrac{2}{3}\right) (0.624)(3.93 \text{ ft}) \sqrt{(2)\left(32.2 \frac{\text{ft}}{\text{sec}^2}\right)} (0.333 \text{ ft})^{\frac{3}{2}} = 2.52 \frac{\text{ft}^3}{\text{sec}}$$

e) Convert discharge into velocity:

$$\text{v} = \frac{Q}{A} = \frac{2.52 \frac{\text{ft}^3}{\text{sec}}}{(4 \text{ ft})(2.5 \text{ ft} + 0.333 \text{ ft})} = 0.222 \text{ ft}/\text{sec}$$

The approach velocity is 0.222 ft/s

Open Channels: Weirs

11) What is the crest length of the Cipolletti weir (USBR standard trapezoidal weir) required to accommodate a flow up to 0.793 m^3/s if the maximum head is limited to 0.259 m?

SOLUTION

Apply the equation for a Cipolletti weir. REMEMBER THAT THIS EQUATION REQUIRES BG UNITS:

$$Q = 3.367bH^{3/2}$$

Units conversion: H = 0.259 m = 0.850 ft

$Q = 0.793\ m^3/sec\ (35.3\ cfs/1\ cms) = 28.0\ cfs$

Apply weir equation to get L :

$$28.0 = 3.367(L)(0.850)^{3/2}$$

Convert L to meters

L = 10.6 ft **= 3.24 m**

Open channels

14) Uniform flow as a depth of 2 meters occurs in a long rectangular channel that is 4 meters wide. The channel is laid on a slope of 0.001, and the Manning coefficient is 0.025. Determine the minimum height of a broad crested weir that can be built on the bottom of this channel to produce critical depth.

SOLUTION

a) Use Manning's equation to obtain the channel discharge and velocity:

$$Q = \frac{1}{n} A R_h^{2/3} S_0^{1/2}$$

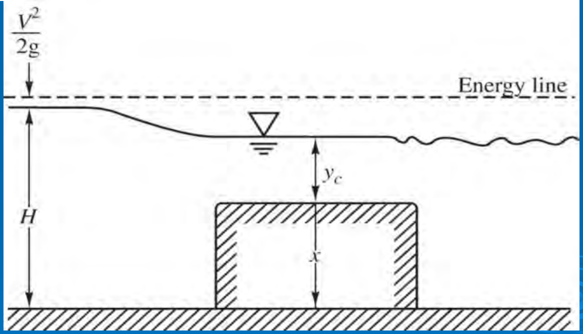
$$Q = \frac{1}{0.025}(8)(1.0)^{2/3}(0.001)^{1/2} = 10.1 \text{ m}^3/\text{sec}$$

$$V = \frac{Q}{A} = \frac{10.1}{8} = 1.26 \text{ m/sec}$$

$$A = (2 \text{ m})(4 \text{ m}) = 8 \text{ m}^2$$

$$P = 2(2 \text{ m}) + 4 \text{ m} = 8 \text{ m}$$

$$R_h = A/P = 1.0 \text{ m}$$



b) Compute specific energy upstream of the weir:

$$E = y + \frac{V^2}{2g} = 2 + \frac{(1.26)^2}{2(9.81)} = 2.08 \text{ m}$$

c) The flow over the weir passes through critical depth; therefore, compute Critical depth:

$$y_c = \sqrt[3]{\frac{Q^2}{gb^2}} = \sqrt[3]{\frac{(10.1)^2}{(9.81)(4)^2}} = 0.87 \text{ m/sec}$$

Open channels

SOLUTION

Obtain the critical velocity corresponding to critical depth:

$$V_c = \frac{Q}{4y_c} = \frac{10.1}{4(0.87)} = 2.90 \text{ m/sec}$$

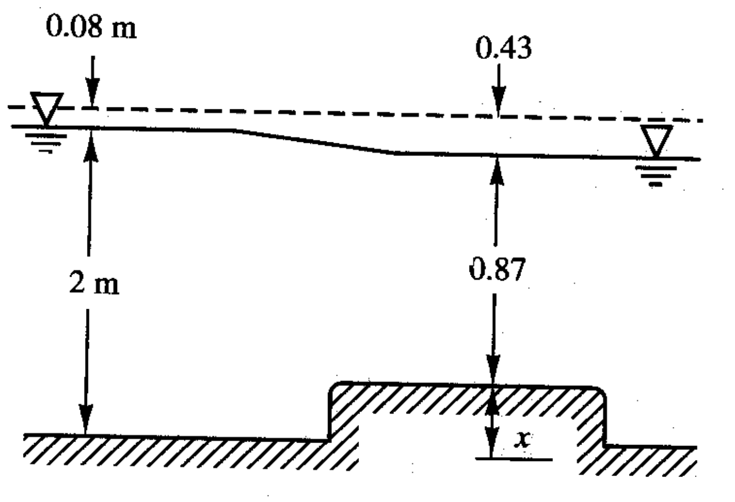
Obtain the critical velocity head. This is the velocity head over the weir:

$$\frac{V_c^2}{2g} = 0.43 \text{ m}$$

Now balance the energy between the upstream channel and the flow over the weir. Assume no energy losses (see figure)

$$E = y_c + \frac{V_c^2}{2g} + x$$

$$2.08 = 0.87 + 0.43 + x$$



Solve for the weir height:

$$x = 0.78 \text{ m}$$

Module 4

Hydraulics: Channel Design

USING UNIFORM FLOW

CHANNEL DESIGN

Dr. Walter F. Silva, UPR

CHANNEL DESIGN

- Open channels are usually designed for uniform flow or normal flow conditions.
- Uniform flow equations are used in sizing these channels
- Design involves: channel alignment, channel size and shape, longitudinal slope and lining material
- Consider several feasible alternatives and compare them to determine the most cost-effective alternative

CHANNEL DESIGN

- Verification of water surface profiles should be done and adjustments must be made to avoid overflows
- Slope stability govern the side slope in natural channels.
- High water table in the soil could be a limitation
- Most channels are designed for subcritical flow

CHANNEL DESIGN
Liners

- Channels are lined to prevent erosion due to shear stresses
- Rigid liners are inflexible such as concrete
- Flexible liners can adjust to soil conditions. Include: gravel, riprap, gabions and grass.

CHANNEL DESIGN
Freeboard

- Freeboard is the vertical distance between the top of the channel and the water surface during design flow conditions
- Take into account variations due to wind, tidal action, flows larger that design flow
- Formula from the Bureau of Reclamation

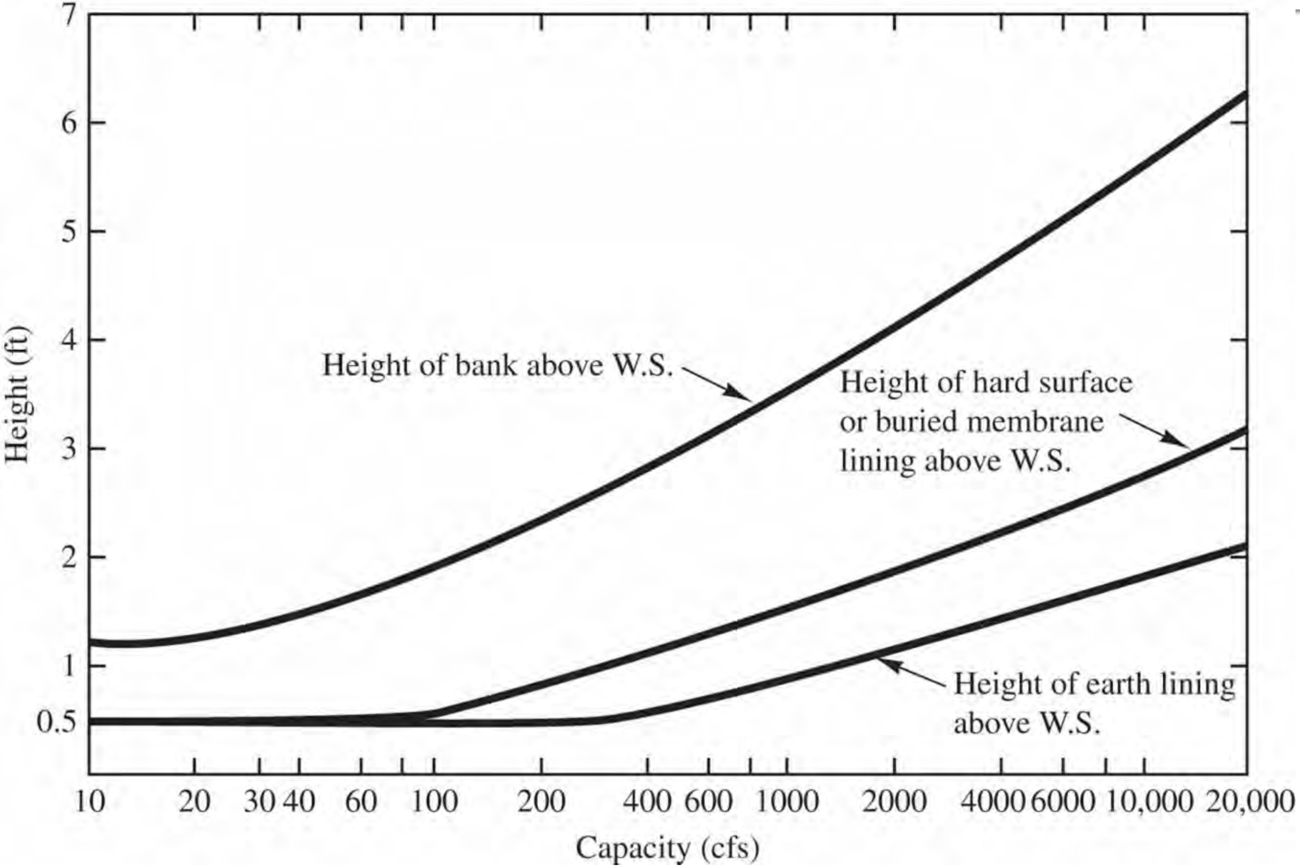
$$F = \sqrt{Cy}$$

CHANNEL DESIGN
Freeboard

$$F = \sqrt{Cy}$$

- F = freeboard
- y = flow depth
- C = Freeboard Coefficient
- F and y in ft ....C = 1.5 if Q = 20 cfs.
- C = 2.5 if Q > 3,000 cfs
- F and y in m ....C = 0.5 if Q = 0.6 cms.
- C = 0.76 if Q > 85 cms

CHANNEL DESIGN
Freeboard



Recommended freeboard and height of banks in lined channels. *Source:* U.S. Bureau of Reclamation, *Linings for Irrigation Canals*, 1976.

CHANNEL DESIGN
Freeboard

- Freeboad distances used in India (Madras Institute of Technology)

Discharge (m^3/s)	<0.15	0.15 – 0.75	0.75 – 1.5	1.5 – 9.0	> 9.0
Freeboard (m)	0.3	0.45	0.60	0.75	0.9

CHANNEL DESIGN

- RIPRAP CHANNELS DESIGN
- GRASS LINED CHANNELS DESIGN
- CONCRETE LINED CHANNELS DESIGN
- GABIONS CHANNEL DESIGN

CHANNEL DESIGN
Unlined Channels

- Two methods are common: Tractive force and maximum permissible velocity
- Maximum velocity refers to the maximum velocity that will not erode the channel
- The maximum permissible velocity depends on the type of material and the channel alignment
- See next table for stable slopes and maximum permissible velocities

CHANNEL DESIGN
Stable lateral slopes for Channels

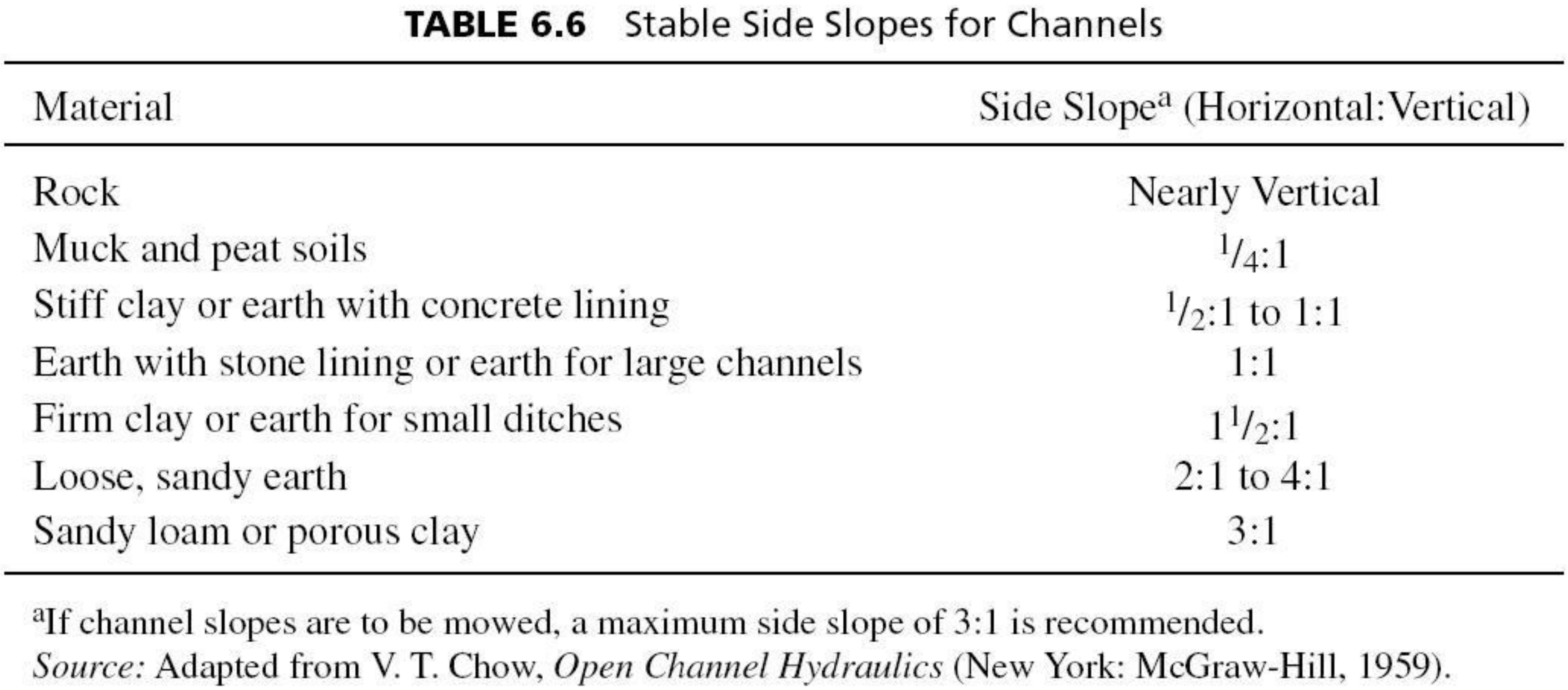


TABLE 6.6 Stable Side Slopes for Channels

Material	Side Slope[a] (Horizontal:Vertical)
Rock	Nearly Vertical
Muck and peat soils	$^{1}/_{4}:1$
Stiff clay or earth with concrete lining	$^{1}/_{2}:1$ to $1:1$
Earth with stone lining or earth for large channels	1:1
Firm clay or earth for small ditches	$1\,^{1}/_{2}:1$
Loose, sandy earth	2:1 to 4:1
Sandy loam or porous clay	3:1

[a] If channel slopes are to be mowed, a maximum side slope of 3:1 is recommended.

Source: Adapted from V. T. Chow, *Open Channel Hydraulics* (New York: McGraw-Hill, 1959).

CHANNEL DESIGN
Maximum permissible velocities

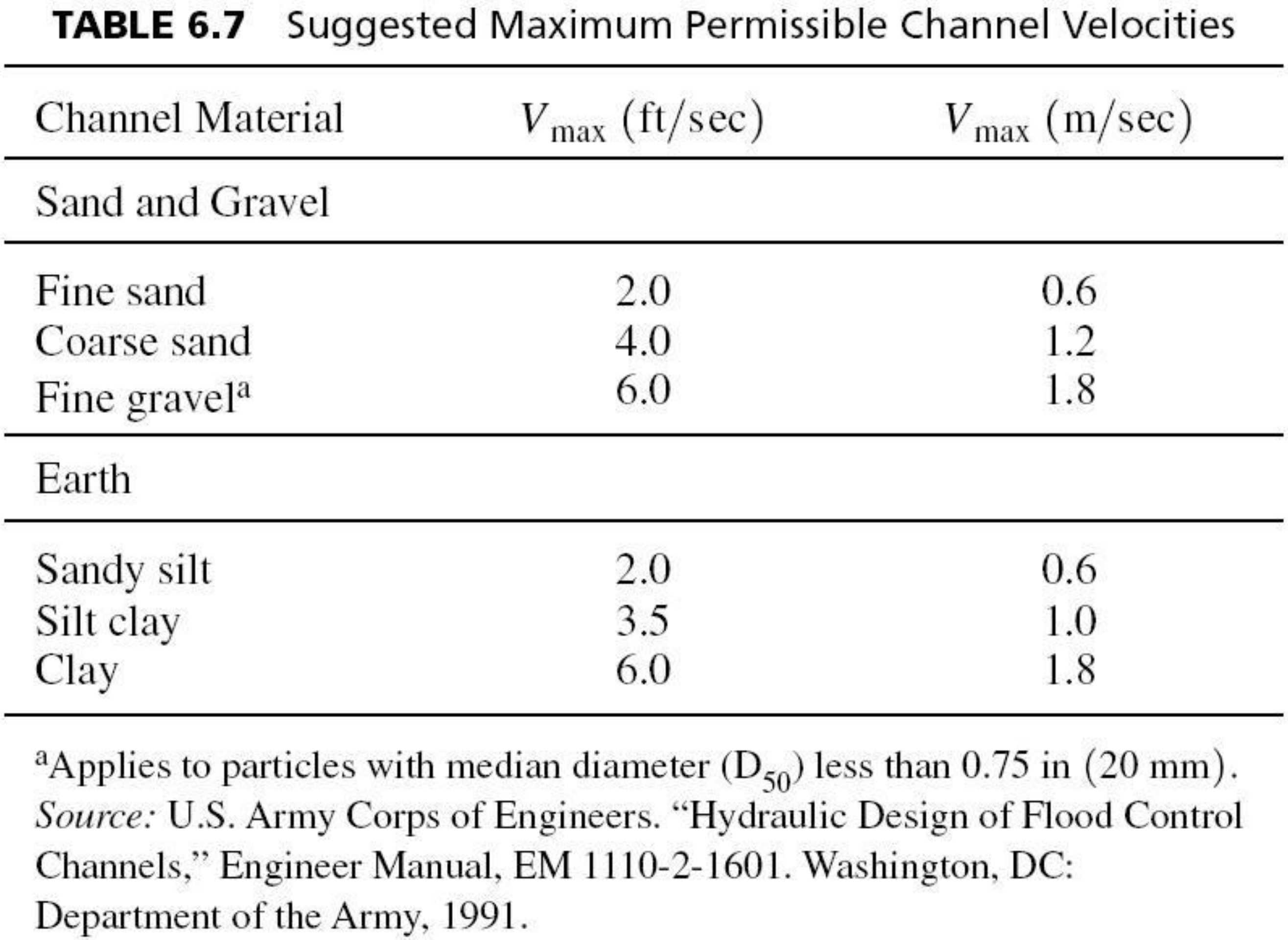


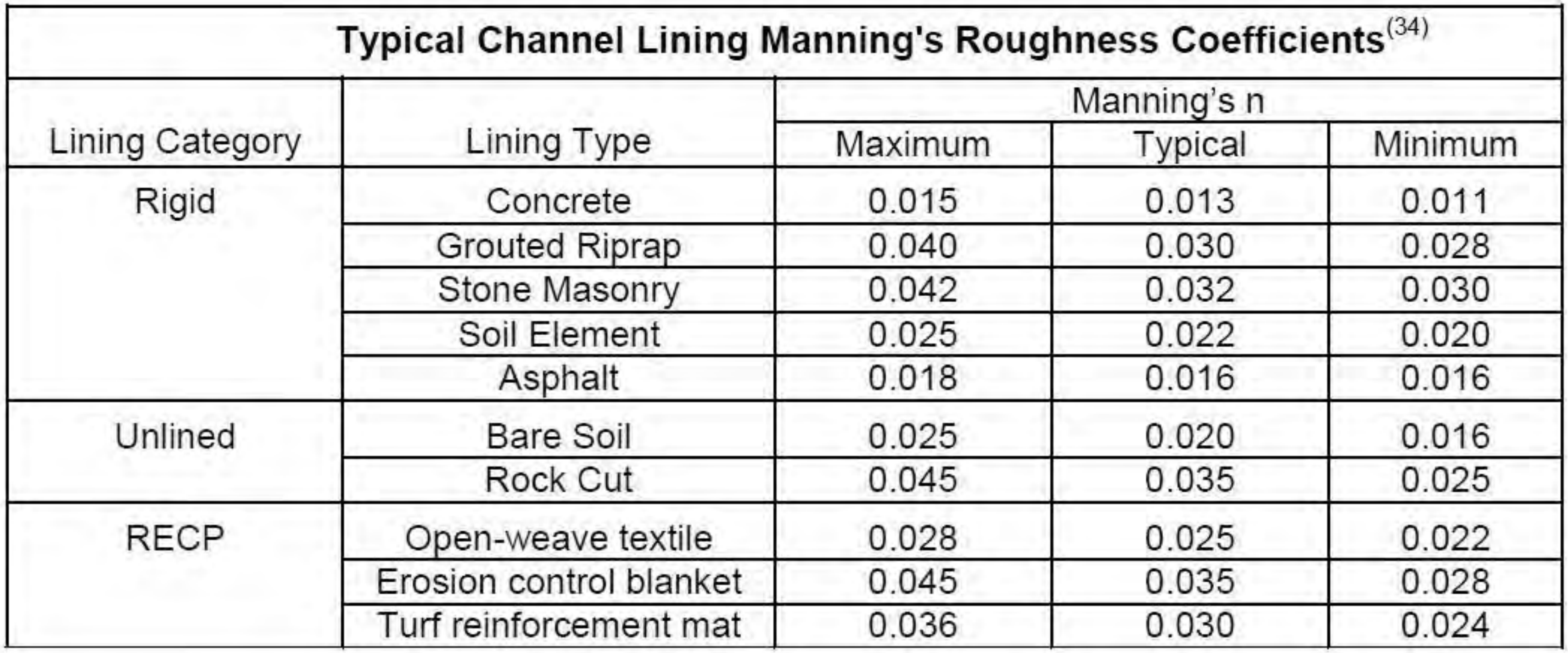
TABLE 6.7 Suggested Maximum Permissible Channel Velocities

Channel Material	V_{max} (ft/sec)	V_{max} (m/sec)
Sand and Gravel		
Fine sand	2.0	0.6
Coarse sand	4.0	1.2
Fine gravel[a]	6.0	1.8
Earth		
Sandy silt	2.0	0.6
Silt clay	3.5	1.0
Clay	6.0	1.8

[a] Applies to particles with median diameter (D_{50}) less than 0.75 in (20 mm).

Source: U.S. Army Corps of Engineers. "Hydraulic Design of Flood Control Channels," Engineer Manual, EM 1110-2-1601. Washington, DC: Department of the Army, 1991.

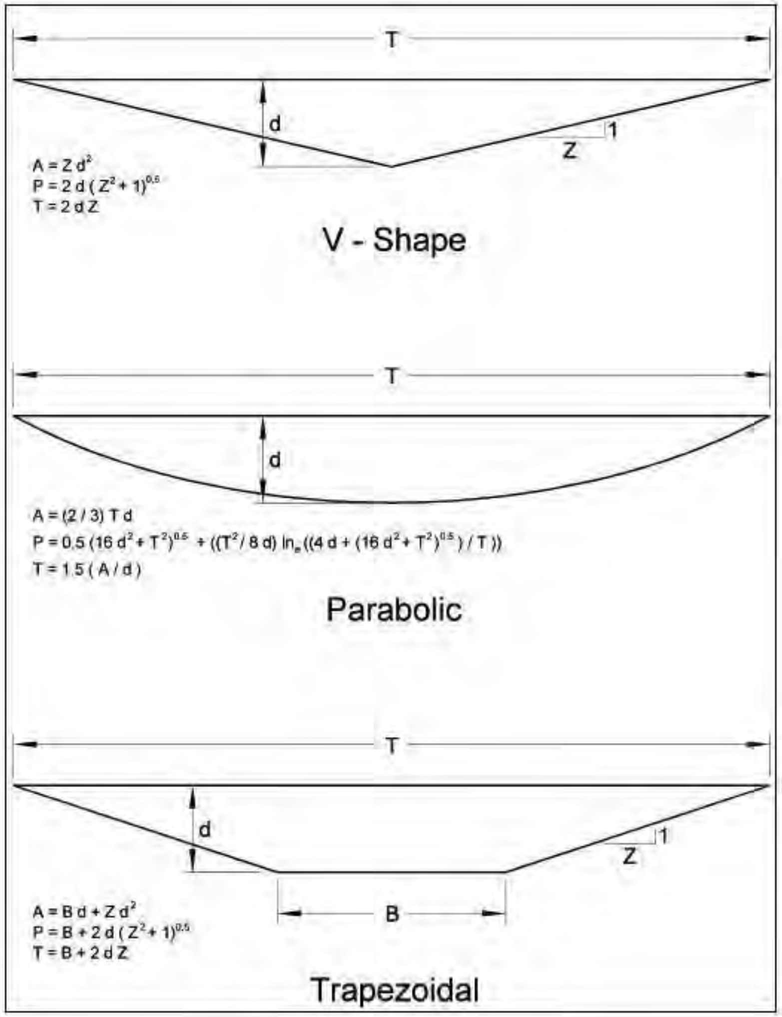
Mannings roughness coefficients for different channel linings



Typical Channel Lining Manning's Roughness Coefficients(34)

Lining Category	Lining Type	Manning's n		
		Maximum	Typical	Minimum
Rigid	Concrete	0.015	0.013	0.011
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Element	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.025	0.020	0.016
	Rock Cut	0.045	0.035	0.025
RECP	Open-weave textile	0.028	0.025	0.022
	Erosion control blanket	0.045	0.035	0.028
	Turf reinforcement mat	0.036	0.030	0.024

Channel Geometries



V - Shape

$A = Z\,d^2$

$P = 2\,d\,(Z^2 + 1)^{0.5}$

$T = 2\,d\,Z$

Parabolic

$A = (2/3)\,T\,d$

$P = 0.5\,(16\,d^2 + T^2)^{0.5} + ((T^2/8\,d)\,\ln_e((4\,d + (16\,d^2 + T^2)^{0.5})/T))$

$T = 1.5\,(A/d)$

Trapezoidal

$A = B\,d + Z\,d^2$

$P = B + 2\,d\,(Z^2 + 1)^{0.5}$

$T = B + 2\,d\,Z$

Channel geometries.

CHANNEL DESIGN

RIPRAP LINING (Guo, 1999 and Sturm, 2001)

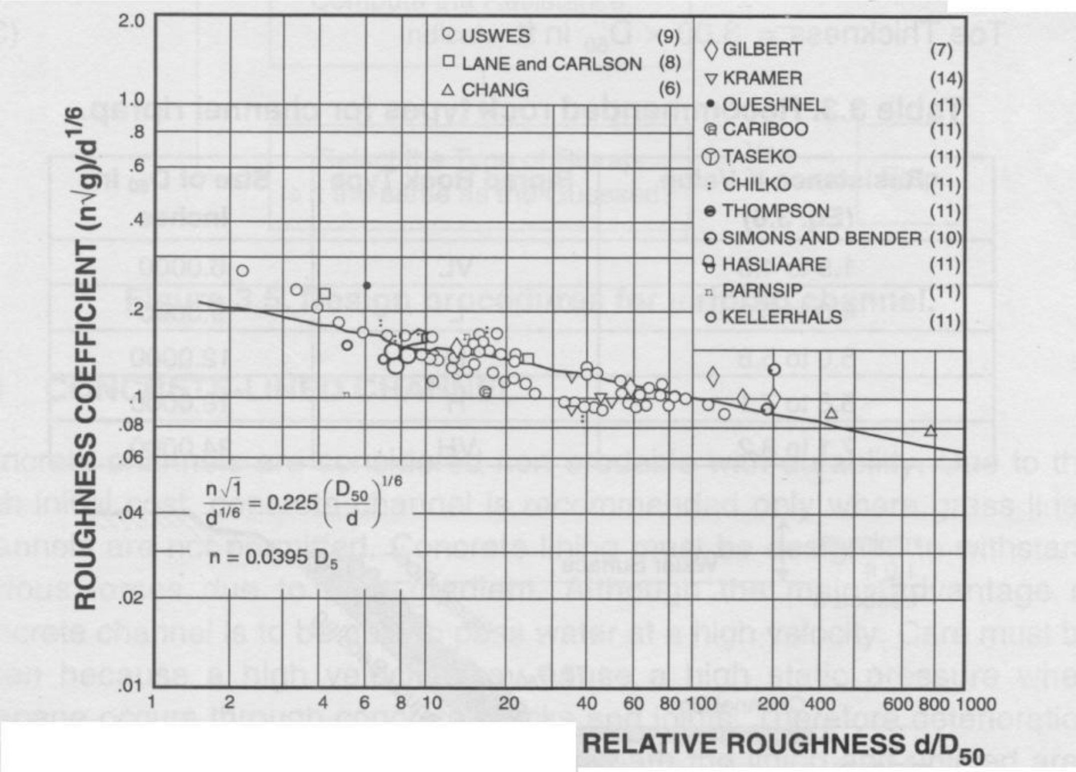
- Riprap is a flexible or adjustable channel lining made of rock with a particular gradation.
- The main design criterion is to choose the channel dimensions and riprap size such that the maximum boundary shear stress does not exceeds the critical shear stress for erosion.
- There are several methods for riprap design. We follow the one proposed by the NCHRP Report 108, 1970.
- Experimental data point out to a relation between the rock characteristics and Manning's *n* given by

$$n = 0.04 D_{50}^{1/6}$$

- This is a form of Strikler's equation

CHANNEL DESIGN

RIPRAP LINING



Variations of Manning's N in a riprap channel.

$$n = 0.04 D_{50}^{1/6}$$

This is a modified Strickler Equation.

Guo, J.C.Y., *Channel Design and Flow Analysis*, WRP, 1999

CHANNEL DESIGN

RIPRAP LINING

- The critical shear stress relation is

$$\tau_{0c} = 4d_{50}$$

- Where τ_{0c} is the critical shear stress required for initiation of motion (lbs/ft2) and d_{50} is the median particle size in feet. This equation is based on Shield's criterion.
- The NCHRP Report 108 adopted the following relations for the maximum shear stress at the bottom and the maximum shear stress on the walls of a trapezoidal channel, respectively as

$$\tau_{o\max} = 1.5\gamma RS \qquad \tau_{0\max}^{w} = 1.2\gamma RS$$

CHANNEL DESIGN

RIPRAP LINING

- The tractive force ratio is defined as:

$$K_r = \frac{\tau_{oc}^{w}}{\tau_{0c}} = \left[1 - \frac{sen^2\theta}{sen^2\phi}\right]^{1/2}$$

  - where θ = side slope angle and ϕ = angle of repose of riprap.
  - τ_{0C}^{w} =critical shear stress on the side wall; and τ_{0C} = critical shear stress for initiation of motion on the bed. These are the shear stresses that causes impending motion on the slope and on the bed.

CHANNEL DESIGN

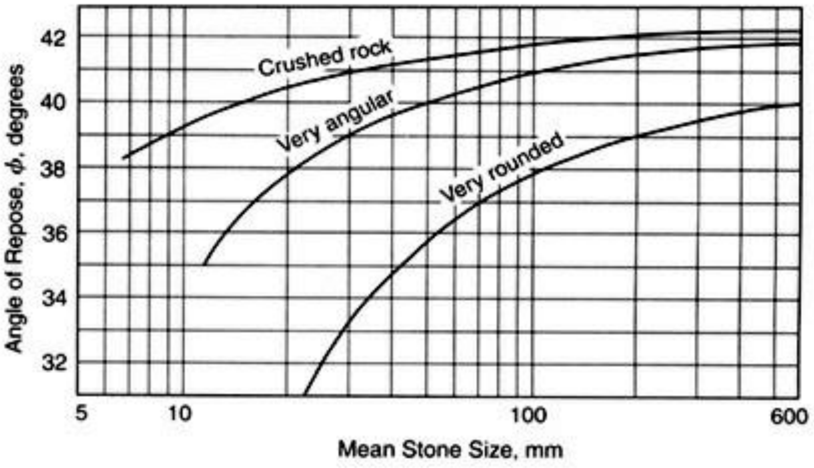
RIPRAP LINING

- K_r is always less than one because a smaller critical shear stress is required to initiate motion on the side slope due to the gravity force component down the slope.
- Notice that the critical shear stress depends upon the lateral slope of the channel and the angle of repose of the riprap.
- The next Figure shows recommended combinations of side slope for different angles of repose of riprap such that the ratios of the maximum shear stress to critical shear stress are approximately equal on the bed and banks.

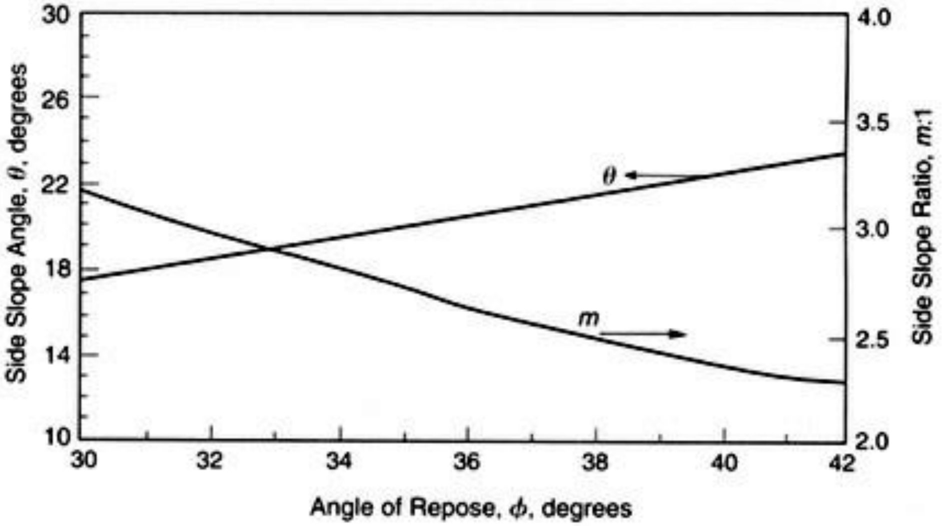
CHANNEL DESIGN

RIPRAP LINING

Angle of repose and recommended channel side slopes for rock riprap (From Sturm, 2001)



(a) Angle of Repose of Riprap



(b) Recommended Side Slopes of Trapezoidal Channels

CHANNEL DESIGN

RIPRAP LINING

- **Design Procedure**

1. Choose a riprap diameter and obtain ϕ and θ from Figure 4.13
$$\tau_{0c} = 4d_{50}$$
2. Calculate the critical bed and wall shear stresses from Equation and the tractive force ratio, Kr equation.

$$\tau_{0c} = 4d_{50} \qquad K_r = \frac{\tau_{oc}^{w}}{\tau_{0c}} = \left[1 - \frac{sen^2\theta}{sen^2\phi}\right]^{1/2} \qquad \tau_{oc}^{w} = K_r \tau_{0c}$$

3. Determine Manning’s n from the modified Strickler’s equation

$$n = 0.04 D_{50}^{1/6}$$

4. For a given channel bottom width, discharge, and slope, find the normal depth from Manning’s equation

CHANNEL DESIGN

RIPRAP LINING

- **Design Procedure**

5. Calculate the maximum bed and side shear stresses using the hydraulic radius and channel slope and compare with the critical values obtained in step 2.

$$\tau_{o\,\mathrm{max}} = 1.5\gamma RS \qquad \tau^{w}_{0\,\mathrm{max}} = 1.2\gamma RS$$

6. If the maximum shear stresses are greater than the critical values, then repeat the process with another riprap diameter. If the maximum shear stresses are just smaller than the critical values; then, proceed to finish the design.

CHANNEL DESIGN

RIPRAP LINING

- **Design Procedure**

7. After having selected the riprap diameter, the thickness of the riprap blanket on the channel bank and its toe are specified as:

Bank Thickness: 1.75 d50 in feet

Toe Thickness: 3.0 d50 in feet

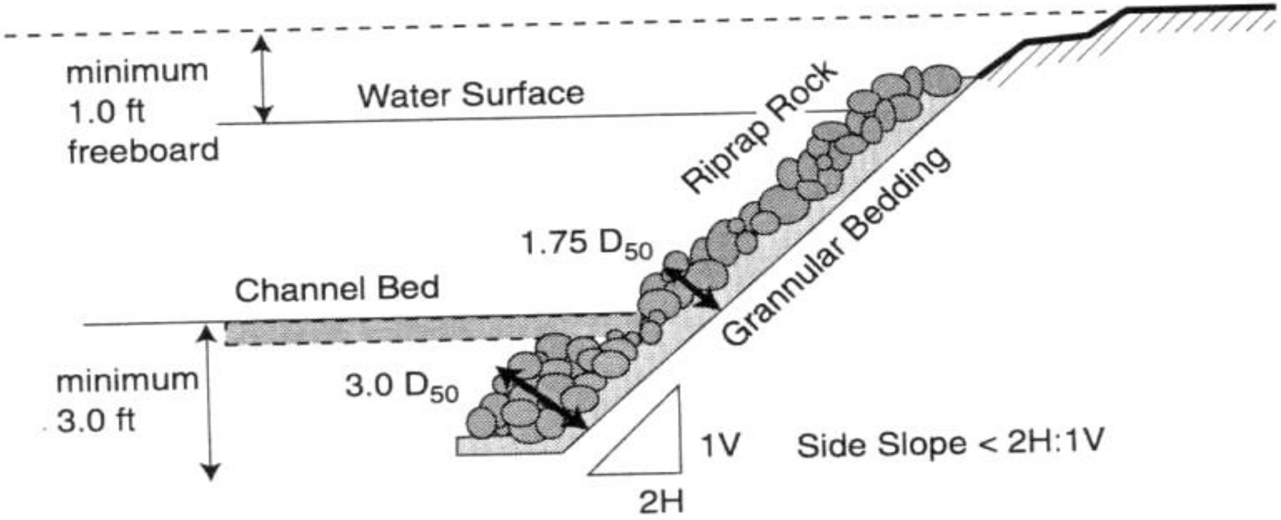


Illustration of riprap blanket on a channel bed.

From Channel Design and Flow Analysis by J.C. Y. Guo, 1999

CONCRETE LINED CHANNELS

- Are considered non-erodable with high durability
- Its initial cost is high
- The concrete mix must resist hydrodynamic forces due to high gradients
- Its major advantage is to be able to pass water at high velocity

CONCRETE LINED CHANNELS Recommendations

- Flow velocities shall not exceed 18 fps for the major discharge
- Adequate freeboard should be provided
- Superelevation of the water surface at bends must be estimated
- The minimum thickness of the concrete lining is 7 in. The side slope could be vertical.

CONCRETE LINED CHANNELS

Manning’s n for different concrete finish

Concrete Surface Finish	*Manning’ s n*
Trowel	0.0130
Float Finish	0.0150
Unfinished	0.0170
Shotcrete, Troweled, or Wavy	0.018-0.020
Shotcrete, Unfinished	0.022

Best Hydraulic Section
Most Efficient Hydraulic Section

- A section that gives maximum discharge, Q, for a specified flow area, A, is called the *most efficient hydraulic section or best hydraulic section.*
- Since Q is proportional to AR2/3 for a given channel, and R = A/P, we can say that the most efficient hydraulic section is the one that yields the minimum wetted perimeter P, for a given A.

Best Hydraulic Section
Most Efficient Hydraulic Section

- Theoretically, the most efficient hydraulic section yields the most economical channel.

- Factors not taken into consideration:
  - Possibility of scour and erosion for erodable channels
  - Amount of overburden in excavation cost
  - Ease of access
  - Transportation of the excavated material
  - Viability of matching cut and fill volumes
  - The cost of lining compared with the cost of excavation
  - Maximum Froude number
  - Right of way and cost of land

CONCRETE LINED CHANNELS Recommendations

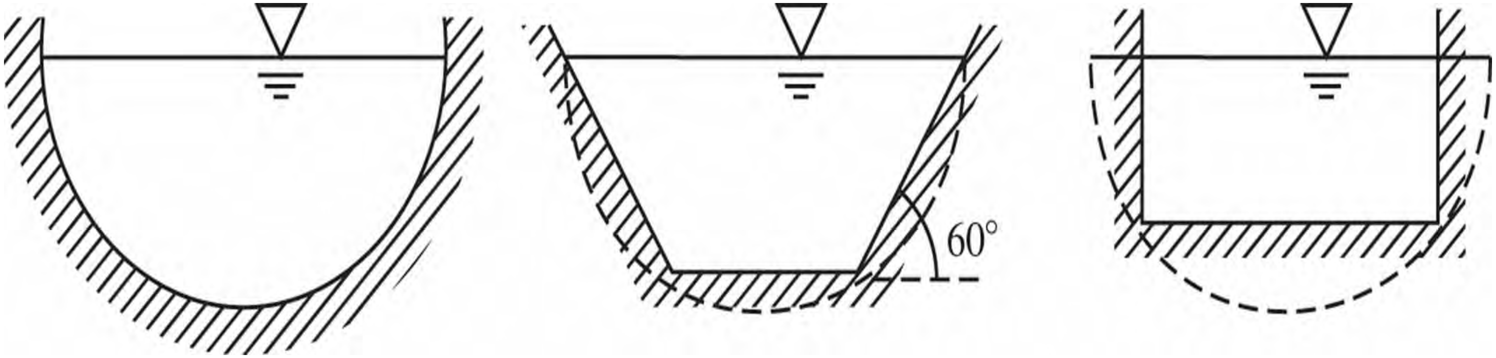
- Flow velocities shall not exceed 18 fps for the major discharge
- Adequate freeboard should be provided
- Superelevation of the water surface at bends must be estimated
- The minimum thickness of the concrete lining is 7 in. The side slope could be vertical.

CONCRETE LINED CHANNELS Recommendations

- The best hydraulic section might be considered for concrete-lined prismatic channels.
- The best hydraulic sections are summarized next

CONCRETE LINED CHANNELS

- Most efficient hydraulic sections



CONCRETE LINED CHANNELS

- Therefore, a rectangular cross section is the most efficient when the flow depth is one-half the channel width.
- For a triangular section, the most efficient cross section has sides inclined at 45 o
- For a trapezoidal channel the most efficient hydraulic section is one-half of a hexagon

Example Lined Channel

- **Given:** A trapezoidal channel with the following characteristics:

$S_o = 0.01$

B = 0.8 m (2.62 ft)

z = 3

d = 0.5 m (1.64 ft)

Find: *The channel capacity and flow velocity if the channel is lined with a turf reinforcement mat.*

Example Lined Channel

Step 1. Determine the channel parameters

Area

$$A = B\,d + 2(1/2)(d)(zd)$$

$$A = B\,d + z\,d^2$$

$$A = (2.62)(1.64) + (3)(1.64)^2$$

$$A = 12.4\ ft^2$$

Perimeter:

$$P = B + 2[(zd)^2 + d^2)]^{0.5}$$

$$P = B + 2d(z^2+1)^{0.5}$$

$$P = (2.62) + (2)(1.64)(3^2+1)^{0.5}$$

$$P = 13.0\ ft$$

Hydraulic Radius

$$R = A/P$$

$$R = 12.4/13.0$$

$$R = 0.95\ ft$$

Example Lined Channel

Step 2. Compute the flow capacity using Manning's Equation

$Qn = K_u A R^{0.67} So^{0.5}$

$Qn = (1.49)(12.4)(0.95)^{0.67}(0.01)^{0.5}$

$Qn = 1.79\ ft^3/s$

$Q = Qn / n$

$Q = 1.79/0.030$

Q= 59.7 ft3/s

Step 3. Compute the flow velocity

$V = Q/A$

$V = 59.7/12.4$

$V = 4.8\ ft/s$

Roadside and Median Channels

- Roadside channels are commonly used with uncurbed roadway sections.
- Median channels are used to prevent drainage from the median areas from running across the travel lanes, slope median areas and inside shoulders to a center swale.
- Median channels are important for high speed facilities and facilities with more than two(2) lanes of traffic in each direction.

[6]

[7]

[8]