P.E. Civil Exam Review:
Hydrology

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C: Groundwater: Well Hydraulics
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Hydrologic Cycle and Terms

- **Precipitation/Rainfall.**
  Discharge of water from the atmosphere.

- **Rainfall excess.**
  Rainfall minus interception, depression storage, evaporation, infiltration.

- **Runoff.**
  Rainfall that appears in surface streams (includes subsurface quick return flow).

- **Surface runoff.**
  Runoff which travels over the soil surface to the nearest stream.
Commonly Used Units

• **Acre-foot:** Volume of water equal to 1 ft. depth of water covering 1.0 acre area = 43,560 ft³.

• **Cfs:** or *cubic feet per second* (ft³/s) - unit of discharge.

• **Cumec:** or *cubic meters per second* (m³/s) - unit of discharge.

• **Cfs-day (sfd):** Volume from a flow of 1 cfs for 1-day (24 hours) = 86,400 cubic feet or 1.98 acre-feet.
### Hydrologic Budget

- **Continuity Equation (Law of Conservation of mass):**

\[
\frac{\Delta S}{\Delta t} = \sum \text{Inflows} - \sum \text{Outflows}
\]

or

\[
\Delta S = S_2 - S_1 = \{ \sum I - \sum Q \} \Delta t
\]

- **Can be applied to:**
  1. Watershed System
  2. Reservoir System
Hydrologic Budget: Watershed System

Example 1: Watershed Runoff Computation

**Given:**
1) Watershed size above a gage site along a river = 50 sq. mi;
2) Watershed annual rainfall = 60 inches/yr;
3) Assume Runoff = 55% of annual rainfall (i.e. 45% losses)

**Compute:**
1) Volume of Annual Runoff from the watershed in acre-feet and sfd
2) Rate of runoff in cfs

**Solution:**

- **Watershed area in**
  - Note: 1 sq. mi = 640 acres

- **Runoff depth in feet**

Annual Volume of runoff = \{50\times640\} \times \{0.55\times60/12\} = 88,000 ac-feet
  - (in 1-year) = 88,000 \times 43,560 = 3833.28 \times 10^6 \text{ ft}^3
  - = 44,366.67 sfd (or cfs-day)

Rate of runoff = \frac{3833.28 \times 10^6 \text{ ft}^3}{365 \times 86400 \text{ s}}
  = 121.55 \text{ cfs}
Hydrologic Budget: Reservoir System

Example 2: Reservoir Storage Computation

Given: During a 30 day period:
1) Streamflow into the reservoir, Q = 5.0 m$^3$/s
2) Water supply withdrawal, W = 136 mgd
3) Evaporation from the reservoir surface = 9.40 cm
4) Average reservoir water surface = 3.75 km$^2$
5) Beginning reservoir storage, $S_1 = 12,560$ ac·ft

Continuity Equation: $S_2 = S_1 + Q - W - E$

(all volume units)

Note: Time period $\Delta t = 30$ days = (30x86,400) sec
Hydrologic Budget: Reservoir System
Example 2: Reservoir Storage Computation (Cont.)

Compute: The month-end reservoir storage, $S_2$ in m$^3$ and ac-ft.

1) $S_1 = (12,560 \text{ ac-ft})$

2) $Q = (5.0 \text{ m}^3/\text{s})(30 \text{ days})(86,400 \text{s/day})$
   \[ = 12,960,000 \text{ m}^3 (35.314 \text{ ft}^3/\text{m}^3)(2.296 \times 10^{-5} \text{ ac-ft/ft}^3) = 10,508 \text{ ac-ft} \]

3) $W = (136 \times 10^6 \text{ gal/day})(0.003785 \text{ m}^3/\text{gal})(30 \text{ days})$
   \[ = 15,442,800 \text{ m}^3 (35.314 \text{ ft}^3/\text{m}^3)(2.296 \times 10^{-5} \text{ ac-ft/ft}^3) = 12,521 \text{ ac-ft} \]

4) $E = (3.75 \text{ km}^2)(1 \times 10^6 \text{ m}^2/\text{km}^2)(9.4 \text{ cm})(0.01 \text{ m/cm})$
   \[ = 352,500 \text{ m}^3 (35.314 \text{ ft}^3/\text{m}^3)(2.296 \times 10^{-5} \text{ ac-ft/ft}^3) = 286 \text{ ac-ft} \]

5) $S_2 = S_1 + Q - W - E$ (for $\Delta t = 30 \text{ days}$) ac-ft
   \[ = (12,560 + 10,508) - (12,521 + 286) \]
   \[ S_2 = 10,261 \text{ ac-ft (loss of storage)} \]  
   (Note: $S_1 = 12,560 \text{ ac-ft}$)

Note: Change in storage, $\Delta S = S_2 - S_1 = (10,261 - 12,560) = -2299 \text{ ac-ft (loss)}$
Problem 1: Reservoir Storage Computation

Given:

- Reservoir located at the outlet of a 150 sq. mile watershed
- Mean annual rainfall, \( P = 38 \) inches (use as inflow into reservoir)
- Mean annual watershed runoff (flow into reservoir), \( Q = 13 \) inches
- Mean annual reservoir evaporation, \( E = 49 \) inches
- Mean daily withdrawal from reservoir (draft), \( D = 100 \) MGD
- Mean reservoir surface area, \( A_s = 4000 \) acres

(Note: 1.0 sq. mi = 640 acres; 1.0 ac-ft = 43,560 ft\(^3\); 1.0 MGD = 3.07 ac-ft/day; 1.0 ft\(^3\) = 7.48 gal; 1.0 cfs = 1.9835 ac-ft/day)

Using a time frame, \( \Delta t = 1 \) year (365 days) determine:

1. Volume of water evaporated from lake in acre-ft/yr:
   a) 16,333   b) 24,586   c) 392,000   d) 55,600

2. Watershed runoff or inflow into reservoir in acre-ft/yr:
   a) 104,000   b) 1,248,000   c) 99,667   d) 266,580

Note: Watershed area not adjusted for reservoir area of 4000 acres
Problem 1: Reservoir Storage Computation (cont.)

3. Watershed runoff or inflow into reservoir in cfs:
   a) 250.8          b) 143.7          c) 550.0          d) 85.6

4. Volume of rainfall input, P, to reservoir in acre-ft/yr
   a) 152,000       b) 304,000       c) 12,667       d) 85,000

5. Mean draft, D in ac-ft/yr:
   a) 100,000       b) 112,055       c) 185,250       d) 265,500

6. Net loss/gain of reservoir storage, ΔS in acre-ft/yr:
   a) -16,280       b) 12,500       c) -11,721       d) 0
Computation of Critical (Maximum) Reservoir Storage

• **Graphical Method:**
  Mass Curve Analysis (Rippl Method)

• **Analytical Method:**
  Sequent Peak Method
Example 3: Estimation of Critical or Maximum Reservoir Storage (Capacity)

Problem: The following mean monthly flows were measured in million gallons/month (MG/month) at a gage in a river during a 13 month critical drought period. Determine the maximum reservoir storage required in MG to provide a constant draft of 35 MG/month (Note: 1.0 MGD = 1.55 cfs or 1.0 MG/month = 46.5 cfs based on 30 days)

<table>
<thead>
<tr>
<th>Month</th>
<th>Inflow, Q MG/month</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>54</td>
</tr>
<tr>
<td>3</td>
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<td>28</td>
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<tr>
<td>9</td>
<td>42</td>
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<tr>
<td>10</td>
<td>108</td>
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<tr>
<td>11</td>
<td>98</td>
</tr>
<tr>
<td>12</td>
<td>22</td>
</tr>
<tr>
<td>13</td>
<td>50</td>
</tr>
</tbody>
</table>
Example 3: Mass Curve Analysis for Estimating Critical Reservoir Capacity - Rippl Method (cont.)

Required Critical Reservoir Storage (Capacity) = (350 - 230) = 120 MG
Example 3: Analytical Solution (Sequent Peak Method) (cont.)

<table>
<thead>
<tr>
<th></th>
<th></th>
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<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>40</td>
<td>35</td>
<td>40</td>
<td>35</td>
<td>5</td>
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<tr>
<td>2</td>
<td>54</td>
<td>35</td>
<td>94</td>
<td>70</td>
<td>24</td>
</tr>
<tr>
<td>3</td>
<td>90</td>
<td>35</td>
<td>184</td>
<td>105</td>
<td>79 P1</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
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<td>194</td>
<td>140</td>
<td>54</td>
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<td>5</td>
<td>7</td>
<td>35</td>
<td>201</td>
<td>175</td>
<td>26</td>
</tr>
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<td>6</td>
<td>8</td>
<td>35</td>
<td>209</td>
<td>210</td>
<td>-1</td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>35</td>
<td>211</td>
<td>245</td>
<td>-34</td>
</tr>
<tr>
<td>8</td>
<td>28</td>
<td>35</td>
<td>239</td>
<td>280</td>
<td>-41 T1</td>
</tr>
<tr>
<td>9</td>
<td>42</td>
<td>35</td>
<td>281</td>
<td>315</td>
<td>-34</td>
</tr>
<tr>
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<td>108</td>
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<td>98</td>
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<td>487</td>
<td>385</td>
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<td>12</td>
<td>22</td>
<td>35</td>
<td>509</td>
<td>420</td>
<td>89 T2</td>
</tr>
<tr>
<td>13</td>
<td>50</td>
<td>35</td>
<td>559</td>
<td>455</td>
<td>104 P3</td>
</tr>
</tbody>
</table>

Required Reservoir Storage = Max{ (P1 - T1) or (P2 - T2) }
= (79 - (-41)) or (102 - 89)
= 120 MG
Two types of rainfall events:

1) HISTORICAL:

- Spatially and temporally averaged hyetographs rainfall events
- Based on measured rainfall depths using rain gages at a point

![Hytograph Diagram]

A 12-hour historical rainfall event
Incremental depths:
Time (h): 0-3 3-6 6-9 9-12
Rainfall (cm): 0.50 0.80 1.80 0.40
Intensity (cm/hr): 0.17 0.27 0.60 0.13

2) DESIGN (or SYNTHETIC):

- Standardized temporal rainfall distributions (design storm hyetographs)
- Based on regionalized historical rainfall data for select duration and frequency
Design Rainfall
Intensity-Duration-Frequency (IDF)

Requires the following:

1) Frequency (F_R) or average return period (T) (see Appendix A)

   Example: For T = 100 years, F_R = 1/T = 1/100 = 0.01 or 1%

2) Duration, t_d

   Usually assumed equal to time of concentration, t_c

3) Design rainfall depth = (intensity x duration)

   Obtained from:

   a) NWS TP 40 Regionalized Rainfall
   b) From Intensity Duration Frequency (IDF) Curves or Equations

4) Time (or temporal) distribution of rainfall depth

5) Spatial variation rainfall depth over a catchment

   Handled using an average rainfall depth
Figure: Regionalized IDF Map for IDF Equations

Map showing areas of approximately similar rainfall characteristics.
<table>
<thead>
<tr>
<th>Frequency years</th>
<th>Area 1</th>
<th>Area 2</th>
<th>Area 3</th>
<th>Area 4</th>
<th>Area 5</th>
<th>Area 6</th>
<th>Area 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>(i = \frac{5230}{t + 30})</td>
<td>(i = \frac{3550}{t + 21})</td>
<td>(i = \frac{2590}{t + 17})</td>
<td>(i = \frac{1780}{t + 13})</td>
<td>(i = \frac{1780}{t + 16})</td>
<td>(i = \frac{1730}{t + 14})</td>
<td>(i = \frac{810}{t + 11})</td>
</tr>
<tr>
<td>5</td>
<td>(i = \frac{6270}{t + 29})</td>
<td>(i = \frac{4830}{t + 25})</td>
<td>(i = \frac{3330}{t + 19})</td>
<td>(i = \frac{2460}{t + 16})</td>
<td>(i = \frac{2060}{t + 13})</td>
<td>(i = \frac{1900}{t + 12})</td>
<td>(i = \frac{1220}{t + 12})</td>
</tr>
<tr>
<td>10</td>
<td>(i = \frac{7620}{t + 36})</td>
<td>(i = \frac{5840}{t + 29})</td>
<td>(i = \frac{4320}{t + 23})</td>
<td>(i = \frac{2820}{t + 16})</td>
<td>(i = \frac{2820}{t + 17})</td>
<td>(i = \frac{3100}{t + 23})</td>
<td>(i = \frac{1520}{t + 13})</td>
</tr>
<tr>
<td>25</td>
<td>(i = \frac{8300}{t + 33})</td>
<td>(i = \frac{6600}{t + 32})</td>
<td>(i = \frac{5840}{t + 30})</td>
<td>(i = \frac{4320}{t + 27})</td>
<td>(i = \frac{3300}{t + 17})</td>
<td>(i = \frac{3940}{t + 26})</td>
<td>(i = \frac{1700}{t + 10})</td>
</tr>
<tr>
<td>50</td>
<td>(i = \frac{8000}{t + 28})</td>
<td>(i = \frac{8890}{t + 38})</td>
<td>(i = \frac{6350}{t + 27})</td>
<td>(i = \frac{4750}{t + 24})</td>
<td>(i = \frac{4750}{t + 25})</td>
<td>(i = \frac{4060}{t + 21})</td>
<td>(i = \frac{1650}{t + 8})</td>
</tr>
<tr>
<td>100</td>
<td>(i = \frac{9320}{t + 33})</td>
<td>(i = \frac{9520}{t + 36})</td>
<td>(i = \frac{7370}{t + 31})</td>
<td>(i = \frac{5590}{t + 28})</td>
<td>(i = \frac{6100}{t + 29})</td>
<td>(i = \frac{5330}{t + 26})</td>
<td>(i = \frac{1960}{t + 10})</td>
</tr>
</tbody>
</table>
Example 4: Estimate the 100-yr, 1 hour design rainfall for Louisville, KY

a) From the IDF Curves for Louisville, Kentucky:
   Rainfall Intensity, \( i = 2.8 \text{ in/hr} \)
   Design Rainfall depth = \( i \times t_d = 2.8 \times 1.0 = 2.8 \text{ inches} \)
Example 4: Estimate the 100-yr, 1 hour design rainfall for Louisville, KY (cont.)

b) From the IDF Map and Table (Slides 17 and 18) Louisville is in Region 3.

Regional IDF equation for T = 100 yr:
\[ i \text{ (mm/hr)} = \frac{7370}{(t+31)} \text{ where } t = t_d \text{ in minutes} \]
\[ i = \frac{7370}{60+31} \]
\[ = 80.989 \text{ mm/hr} \]
\[ = 3.2 \text{ in/hr} \]

Rainfall Depth = 3.2 in

From Louisville IDF Curve P = 2.80 in (see previous slide)

• Note the difference between using a locally developed IDF curve versus using a regional equation

Determine:

1. The 100-year, 60 min rainfall intensity for Chicago in in/hr is:
   a) 2.5; b) 3.2; c) 3.0; d) 6.0

2. The 100-year, 30 min rainfall intensity (in in/hr) using the Chicago IDF Equation
   \( \text{Equation i (mm/hr)} = \frac{7370}{(t+31)} \) (from region 3; see slides 17 and 18):
   a) 4.5; b) 3.5; c) 2.8; d) 4.8

3. The 100-year, 30 min rainfall depth in inches is:
   a) 3.0; b) 4.0; c) 2.4; d) 1.5
Effective Rainfall (or rainfall excess), $P_e$ equals

Rainfall (P) minus

Abstractions due to:

- Infiltration
- Depression Storage
- Interception

Rainfall Excess = Volume of Direct Runoff, $Q$
Abstractions (Cont.)

- **Main Abstraction Process is Infiltration**

- **Interception and Depression Storage**
  occur in the early stages of event and can be considered as initial losses

- **Some methods for infiltration:**
  - Infiltration Indices: $\Phi$ index
  - Runoff Coefficients: Rational Method – $C$
  - SCS Curve Number Method, $CN$
  - Infiltration Capacity Curves: Horton's
Example 5: Rainfall Excess and Volume of Direct Runoff

Given:

- A direct runoff hydrograph from a 7.0 sq. mi watershed.
- A uniform loss rate, \( \Phi = 0.50 \text{ in/hr} \) (or 0.25 in/1/2hr).
- Calculate volume of direct runoff in inches.
- Show volume of direct runoff = volume of rainfall excess

Note: Rainfall intensity, \( i > \Phi \) in all time intervals.

<table>
<thead>
<tr>
<th>Time Period (1/2 hr)</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall Intensity, ( i ) (in/hr):</td>
<td>2.7</td>
<td>4.30</td>
<td>4.10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Direct Runoff, ( Q ) (cfs):</td>
<td>430</td>
<td>1920</td>
<td>5300</td>
<td>9130</td>
<td>10625</td>
<td>7830</td>
<td>3920</td>
<td>1845</td>
<td>1400</td>
<td>830</td>
<td>310</td>
</tr>
</tbody>
</table>
Example 5: Rainfall Excess and Volume of Direct Runoff (cont.)

Solution:
Volume of Rainfall Excess, \( P_e \)

\[
= \{(2.70-0.50)\times0.5\} + \{(4.30-0.50)\times0.5\} + \{(4.1-0.50)\times0.5\}
\]

\[
= 4.8 \text{ inches}
\]

Volume of Direct Runoff, \( V_d = \sum Q_n \times \Delta t \)

\[
= (430 + 1920 + 5300 + 9130 + 10625 + 7830 + 3920 + 1845 + 1400 + 830 + 310) \times 0.5
\]

\[
= (43,540 \times 0.5) = 21,770 \text{ cfs-hrs}
\]

\[
= (21,770 \times 3600) = 78,372,000 \text{ ft}^3
\]

Direct Runoff Depth, \( r_d \) (inches)

\[
= \frac{\text{direct runoff volume in ft}^3}{\text{drainage area in ft}^2}
\]

\[
= 78,372,000 / (7 \times 5280^2) = 0.40 \text{ ft.}
\]

\[
= 4.8 \text{ inches}
\]

Note: Rainfall Excess Depth, \( P_e \) = Direct Runoff Depth, \( r_d \) = 4.8 in.
Runoff Methods

1. Peak Discharge, $Q_p$, Methods:
   
   - Rational Method
   
   - SCS Curve Number Method
     (and TR 55 Graphical Peak Discharge Method)

2. Unit Hydrograph Method
Runoff – Small Catchments

**Rational Method**

- Ideal for small catchments (less than 100 acres)
- Used widely in Urban Storm water sewer design
- Requires the following design variables:
  - a) Runoff Coefficient C (refer to slide 29)
  - b) Maximum Rainfall intensity, i (in in/hr or mm/hr) (obtained from IDF Curves or IDF Equations for a specified duration usually equal to time of concentration, $t_c$)
  - c) Time of Concentration, $t_c$
  - d) Catchment Area, A (in acres or km$^2$ or Hectares (Ha))

 Refer to Slides 17 and 18
Rational Method-Peak Flow Formulae

• **US units:** \( Q_p = C \times i \times A \) (in cfs)

- \( i \) in in/hr
- \( A \) in acres

• **SI units:** \( Q_p = 0.278 \times C \times i \times A \) (in m\(^3\)/s)

- \( i \) in mm/hr
- \( A \) in km\(^2\)

- \( Q_p = 2.78 \times C \times i \times A \) (SI units) (in liters/s),

- \( i \) in mm/hr
- \( A \) in Ha

**Note:** 1.0 Ha = 2.471 acres = 10,000 m \(^2\) ; 1.0 km\(^2\) = 100 Ha
Table: Runoff Coefficients, C
For the Rational Method:

<table>
<thead>
<tr>
<th>Character of surface</th>
<th>2</th>
<th>5</th>
<th>10</th>
<th>25</th>
<th>50</th>
<th>100</th>
<th>500</th>
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</thead>
<tbody>
<tr>
<td>Developed</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Asphaltic</td>
<td>0.73</td>
<td>0.77</td>
<td>0.81</td>
<td>0.86</td>
<td>0.90</td>
<td>0.93</td>
<td>1.00</td>
</tr>
<tr>
<td>Concrete/roof</td>
<td>0.75</td>
<td>0.80</td>
<td>0.83</td>
<td>0.88</td>
<td>0.92</td>
<td>0.97</td>
<td>1.00</td>
</tr>
<tr>
<td>Grass areas (lawns, parks, etc.)</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Poor condition (grass cover less than 50% of the area)</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Flat, 0–2%</td>
<td>0.32</td>
<td>0.34</td>
<td>0.37</td>
<td>0.40</td>
<td>0.44</td>
<td>0.47</td>
<td>0.58</td>
</tr>
<tr>
<td>Average, 2–7%</td>
<td>0.37</td>
<td>0.40</td>
<td>0.43</td>
<td>0.46</td>
<td>0.49</td>
<td>0.53</td>
<td>0.61</td>
</tr>
<tr>
<td>Steep, over 7%</td>
<td>0.40</td>
<td>0.43</td>
<td>0.45</td>
<td>0.49</td>
<td>0.52</td>
<td>0.55</td>
<td>0.62</td>
</tr>
<tr>
<td>Fair condition (grass cover on 50% to 75% of the area)</td>
<td></td>
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<td></td>
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</tr>
<tr>
<td>Flat, 0–2%</td>
<td>0.25</td>
<td>0.28</td>
<td>0.30</td>
<td>0.34</td>
<td>0.37</td>
<td>0.41</td>
<td>0.53</td>
</tr>
<tr>
<td>Average, 2–7%</td>
<td>0.33</td>
<td>0.36</td>
<td>0.38</td>
<td>0.42</td>
<td>0.45</td>
<td>0.49</td>
<td>0.58</td>
</tr>
<tr>
<td>Steep, over 7%</td>
<td>0.37</td>
<td>0.40</td>
<td>0.42</td>
<td>0.46</td>
<td>0.49</td>
<td>0.53</td>
<td>0.60</td>
</tr>
<tr>
<td>Good condition (grass cover larger than 75% of the area)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat, 0–2%</td>
<td>0.21</td>
<td>0.23</td>
<td>0.25</td>
<td>0.29</td>
<td>0.32</td>
<td>0.36</td>
<td>0.49</td>
</tr>
<tr>
<td>Average, 2–7%</td>
<td>0.29</td>
<td>0.32</td>
<td>0.35</td>
<td>0.39</td>
<td>0.42</td>
<td>0.46</td>
<td>0.56</td>
</tr>
<tr>
<td>Steep, over 7%</td>
<td>0.34</td>
<td>0.37</td>
<td>0.40</td>
<td>0.44</td>
<td>0.47</td>
<td>0.51</td>
<td>0.58</td>
</tr>
<tr>
<td>Undeveloped</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cultivated Land</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat, 0–2%</td>
<td>0.31</td>
<td>0.34</td>
<td>0.36</td>
<td>0.40</td>
<td>0.43</td>
<td>0.47</td>
<td>0.57</td>
</tr>
<tr>
<td>Average, 2–7%</td>
<td>0.35</td>
<td>0.38</td>
<td>0.41</td>
<td>0.44</td>
<td>0.48</td>
<td>0.51</td>
<td>0.60</td>
</tr>
<tr>
<td>Steep, over 7%</td>
<td>0.39</td>
<td>0.42</td>
<td>0.44</td>
<td>0.48</td>
<td>0.51</td>
<td>0.54</td>
<td>0.61</td>
</tr>
<tr>
<td>Pasture/Range</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat, 0–2%</td>
<td>0.25</td>
<td>0.28</td>
<td>0.30</td>
<td>0.34</td>
<td>0.37</td>
<td>0.41</td>
<td>0.53</td>
</tr>
<tr>
<td>Average, 2–7%</td>
<td>0.33</td>
<td>0.36</td>
<td>0.38</td>
<td>0.42</td>
<td>0.45</td>
<td>0.49</td>
<td>0.58</td>
</tr>
<tr>
<td>Steep, over 7%</td>
<td>0.37</td>
<td>0.40</td>
<td>0.42</td>
<td>0.46</td>
<td>0.49</td>
<td>0.53</td>
<td>0.60</td>
</tr>
<tr>
<td>Forest/Woodlands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat, 0–2%</td>
<td>0.22</td>
<td>0.25</td>
<td>0.28</td>
<td>0.31</td>
<td>0.35</td>
<td>0.39</td>
<td>0.48</td>
</tr>
<tr>
<td>Average, 2–7%</td>
<td>0.31</td>
<td>0.34</td>
<td>0.36</td>
<td>0.40</td>
<td>0.43</td>
<td>0.47</td>
<td>0.56</td>
</tr>
<tr>
<td>Steep, over 7%</td>
<td>0.35</td>
<td>0.39</td>
<td>0.41</td>
<td>0.45</td>
<td>0.48</td>
<td>0.52</td>
<td>0.58</td>
</tr>
</tbody>
</table>
Time of Concentration, $t_c$

**Definition:**

$$t_c = \sum \text{travel times from the hydraulically remotest point in a catchment}$$

$$= (\text{Overland flow time}) + (\text{Channel or Pipe flow) time}$$

Refer to Appendix A for Methods for Computing Time of Concentration

Example: $t_c$ at this outlet

$$= \max (t_1 + t_2) \text{ or } t_3 \text{ or } t_4$$
Example 6: Rational Method for Catchment Runoff Peak

- Two watersheds A and B (located in Tallahassee, Florida) drain at a common outlet. Determine the peak outflow, \( Q_p \), in cfs for a design return period \( T = 25 \) years. Also calculate the size of a circular concrete pipe (Manning’s \( n = 0.025 \), slope = 0.005) which can handle the peak flow. IDF Equation for Tallahassee, Florida, \( i \) (in mm/hr) \( = \{ \frac{8300}{(t+33)} \} \) with \( t \) in min.

\[
\begin{align*}
A &= 100 \text{ acres} \\
C_1 &= 0.42 \\
t_1 &= 20 \text{ min} \\
B &= 200 \text{ acres} \\
C_2 &= 0.38 \\
t_2 &= 60 \text{ min}
\end{align*}
\]
Example 6: Rational Method for Catchment Runoff Peak (cont.)

- Composite $C_c = (C_1A_1 + C_2A_2)/(A_1+A_2)$
- $= (0.42 \times 100 + 0.38 \times 200)/(100+200) = 0.39$

Note for multiple areas:

$$Q_p = C_c i A_T = (\sum C_i i / \sum A_i) A_T = i \sum C_i A_i$$

where total area, $A_T = \sum A_i$

- Time of Concentration, $t_c = \max (t_1, t_2) = \max (20, 60) = 60$ min

Tallahassee, FL, IDF Eq. Area 1 (slide 18)

- 25-year Rainfall intensity $i$ for $t_c = 60$ min $= 3.5$ in/hr

Rational Method Formula (US units)

- Peak Flow $Q_p = C_c i A = 0.39 \times 3.5 \times 300 = 409.5$ cfs

From Manning’s (US Units)

$$D = (2.16 \times Q_p n / \sqrt{S_o})^{3/8}$$

- Pipe diameter, $D = \{ 2.16 \times 409.5 \times 0.020 / (0.005)^{0.5} \}^{3/8} = 7.93$ feet

$= 95.2$ inches (Use 96 inch pipe)
Problem 3: Application of Rational Method in Storm Sewer Design

Given:
IDF Equation for the area is:
\[ i \text{ (in/hr)} = \frac{120T^{0.175}}{(t_d + 27)} \]
where \( T \) = Return Period in years
\( t_d \) = Storm duration = time of concentration, \( t_c \) (in minutes)

Determine Peak Flow from Sub-area Area III and Pipe size EB Using Rational Method

Elv. E = 498.43 ft; Elv. B = 495.55
Length of pipe EB = 450 ft
Problem 3: Application of Rational Method in Storm Sewer Design (cont.)

Answer the following questions for a rainfall event with a return period $T = 5$-yrs:

1) The maximum rainfall intensity $i$ in in/hr is:
   a) 2.5;  b) 5.2;  c) 4.3;  d) 3.5

2) The peak flow $Q_p$ (in cfs) using rational method formula from Area III into inlet E is:
   a) 5.5;  b) 10.3;  c) 15.2;  d) 75.5

3) The slope $S_0$ of pipe EB is:
   a) 0.005;  b) 0.00034;  c) 0.0002;  d) 0.0064

4) The required diameter of sewer pipe EB (in inches) to handle peak flow $Q_p$ is (assume $n = 0.015$):
   a) 15.2;  b) 32.6;  c) 25.8;  d) 20.5

From Manning’s (US Units)
$D = (2.16xQ_p n/\sqrt{S_o})^{3/8}$
Problem 3: Application of Rational Method in Urban Storm Sewer Design- Complete Solution (cont.)

<table>
<thead>
<tr>
<th>Sewer pipe</th>
<th>2 Length</th>
<th>3 Slope $S_o$</th>
<th>4 Total area drained (acres)</th>
<th>5 $\sum CA$</th>
<th>6 $t_c$ (min)</th>
<th>7 Rainfall intensity $i$ (in/hr)</th>
<th>8 Design discharge $Q$ (cfs)</th>
<th>9 Computed sewer diameter (ft)</th>
<th>10 Pipe size used (ft)</th>
<th>11 Flow velocity $Q/A$ (ft/s)</th>
<th>12 Flow time $L/V$ (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EB</td>
<td>450</td>
<td>0.0064</td>
<td>4</td>
<td>2.4</td>
<td>10.0</td>
<td>4.68</td>
<td>16.4</td>
<td>1.94</td>
<td>2.00</td>
<td>5.21</td>
<td>1.76</td>
</tr>
<tr>
<td>AB</td>
<td>550</td>
<td>0.0081</td>
<td>5</td>
<td>3.5</td>
<td>7.0</td>
<td>4.09</td>
<td>40.9</td>
<td>2.87</td>
<td>3.00</td>
<td>5.78</td>
<td>1.15</td>
</tr>
<tr>
<td>BC</td>
<td>400</td>
<td>0.0064</td>
<td>18</td>
<td>10.8</td>
<td>15.0</td>
<td>3.79</td>
<td>40.9</td>
<td>2.87</td>
<td>3.00</td>
<td>5.78</td>
<td>1.15</td>
</tr>
<tr>
<td>CD</td>
<td>450</td>
<td>0.0064</td>
<td>27</td>
<td>15.3</td>
<td>16.2</td>
<td>3.68</td>
<td>56.3</td>
<td>3.22</td>
<td>3.50</td>
<td>5.85</td>
<td>1.28</td>
</tr>
</tbody>
</table>

Note for multiple areas:

Peak Flow, $Q = C_c i A_T = (\sum C_i i / \sum A_i) A_T = i \sum C_i A_i$

where total area, $A_T = \sum A_i$
Runoff - Midsize Catchments:

SCS TR55 Method
Runoff - Midsize Catchments: SCS TR55 Method

Requires:

• Cumulative 24-hour Design Rainfall depth, P (in inches) for a selected return period, T (frequency)
Runoff - Midsize Catchments: SCS TR55 Method (cont.)

Requires: SCS Curve Number, CN, based on Land use, Soil, and Antecedent Moisture Condition (AMC)

<table>
<thead>
<tr>
<th>Land Use Description</th>
<th>Hydrologic Soil Group</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>Cultivated land: without conservation treatment</td>
<td>72</td>
</tr>
<tr>
<td>with conservation treatment</td>
<td>62</td>
</tr>
<tr>
<td>Pasture or range land: poor condition</td>
<td>68</td>
</tr>
<tr>
<td>good condition</td>
<td>39</td>
</tr>
<tr>
<td>Meadow: good condition</td>
<td>30</td>
</tr>
<tr>
<td>Wood or forest land: thin stand, poor cover, no mulch</td>
<td>45</td>
</tr>
<tr>
<td>good cover</td>
<td>25</td>
</tr>
<tr>
<td>Open Spaces, lawns, parks, golf courses, cemeteries, etc.</td>
<td>39</td>
</tr>
<tr>
<td>good condition: grass cover on 75% or more of the area</td>
<td>49</td>
</tr>
<tr>
<td>fair condition: grass cover on 50% to 75% of the area</td>
<td>89</td>
</tr>
<tr>
<td>Commercial and business areas (85% impervious)</td>
<td>81</td>
</tr>
<tr>
<td>Industrial districts (72% impervious)</td>
<td>81</td>
</tr>
<tr>
<td>Residential:</td>
<td>81</td>
</tr>
<tr>
<td>Average lot size</td>
<td>77</td>
</tr>
<tr>
<td>1/8 acre or less</td>
<td>65</td>
</tr>
<tr>
<td>1/4 acre</td>
<td>61</td>
</tr>
<tr>
<td>1/3 acre</td>
<td>57</td>
</tr>
<tr>
<td>1/2 acre</td>
<td>54</td>
</tr>
<tr>
<td>1 acre</td>
<td>51</td>
</tr>
<tr>
<td>Paved parking lots, roofs, driveways, etc.</td>
<td>98</td>
</tr>
<tr>
<td>Streets and roads:</td>
<td>98</td>
</tr>
<tr>
<td>paved with curbs and storm sewers</td>
<td>98</td>
</tr>
<tr>
<td>gravel</td>
<td>76</td>
</tr>
<tr>
<td>dirt</td>
<td>72</td>
</tr>
</tbody>
</table>

1 For a more detailed description of agricultural land use curve numbers, refer to Soil Conservation Service, 1972, Chap. 9.
2 Good cover is protected from grazing and litter and brush cover soil.
3 Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.
4 The remaining pervious areas (lawns) are considered to be in good pasture condition for these curve numbers.
5 In some warmer climates of the country a curve number of 95 may be used.
Runoff - Midsize Catchments: SCS TR55 Method (cont.)

Cumulative Direct Runoff Volume (inches), Q

Based on Cumulative Design Rainfall P, total Direct Runoff Volume is computed using an equation or figure shown below:

\[ Q = \frac{(P - I_a)^2}{(P - I_a + S)}; \]

Sorptivity (inches): \[ S = \frac{1000}{CN} - 10 \]

Initial Abstraction (inches): \[ I_a = 0.2 \times S \]

Note: \( P_e \) is same as \( Q \)
Example 7: SCS CN and Direct Runoff Calculations (Chow et al., 1988)

• **Given:**

Rain falls on a 0.05 square miles (32 acres or 12.95 ha) urban catchment with an intensity of 0.85 in/hr (2.16 cm/hr) for a duration of 3 hours. The soil is classified as SCS hydrologic soil group (HSG) C in the entire catchment. **Land use** within the catchment is as follows:

1) 20% area is business district (85% impervious; HSG =C)
2) 80% area is residential district (1/3 acre lots with 30%impervious)

• **Determine total runoff volume in inches and ft³.**
Example 7: SCS CN and Direct Runoff Calculations (cont.) (Chow et al., 1988)

Solution:

1. Determine SCS Composite CN:

   • Business District
     \[ CN = (0.85 \times 98 + 0.15 \times 74) = 94 \]

   • Residential District (1/3 acre lots)
     \[ CN = (0.30 \times 98 + 0.70 \times 74) = 81 \]

   • Composite CN for Catchment
     \[ CN = (0.20 \times 94 + 0.80 \times 81) = 83.6 \]

   Note: In computing Composite CN in urban areas any area not urbanized is assumed to be open space in good hydrologic condition (see SCS Curve Number, CN Table foot-note 4 – Slide 38).

   Note: From CN Table for HSG C (Slide 38): Imperious Area CN = 98
   Open space in good condition CN = 74
Example 7: SCS CN and Direct Runoff Calculations (cont.) (Chow et al., 1988)

2. Compute total rainfall:

\[ P = 0.85 \text{ in/hr} \times 3 \text{ hours} = 2.55 \text{ inches} \]

3. Compute Runoff Volume \( Q \) (in inches):

Sorptivity, \( S = \frac{1000}{\text{CN}} - 10 = \frac{1000}{83.6} - 10 = 1.96 \) inches

Initial Abstraction, \( I_a = 0.2S = 0.2 \times 1.96 = 0.392 \) inches

Direct runoff depth, \( Q = \frac{(P - I_a)^2}{(P - I_a + S)} \)

\[ = \frac{(2.55 - 0.392)^2}{(2.55 - 0.392 + 1.96)} = 1.13 \text{ inches} \]

Direct runoff volume \( V_d = \frac{(1.13/12)}{\text{in}} \times (0.05 \times 640 \times 43,560) \)

\[ = 131,260.8 \text{ ft}^3 \]

\[ = (0.02832 \times 131,260.8) = 3,717.3 \text{ m}^3 \]

Note: 1.0 ft\(^3\) = 0.02832 m\(^3\)
Problem 4: Calculating SCS CN and Direct Runoff, Q

An undeveloped 1000 acre catchment currently is covered by pasture in good condition and is composed of hydrologic soil group C. This gives a *pre-development* composite SCS CN equal to 74

A proposed urban development *(post development)* will change the land use to:

1) 55% 1/3 acre lots (30% impervious), CN = 81;

2) 20% in open space in good condition, CN = 74;

3) 25% in roads, sewers and parking lots, CN = 98.

Note: Refer to SCS CN Table, Slide 38 for curve numbers.
Problem 4: Calculating SCS CN and Direct Runoff, Q (Cont.)

Assuming AMC II condition answer the following questions:

1) The post-development composite SCS CN is:
   a) 90.2   b) 78.6   c) 83.9   d) 89.5

2) The pre-development direct runoff $Q$ associated with a 10-year, 24-hour rainfall of $P = 3.2$ inches is (Note: $CN = 74$):
   (Use Figure – Slide 39 or Equation: $Q = P_e = (P - I_a)^2/(P - I_a + S)$):
   a) 2.5   b) 1.5   c) 1.0   d) 2.1

3) The post-development direct runoff $Q$ associated with a 10-year, 24-hour rainfall of 3.2 inches is:
   (Use Figure – Slide 39 or Equation: $Q = P_e = (P - I_a)^2/(P - I_a + S)$):
   a) 3.2   b) 2.8   c) 1.3   d) 1.7

Note: For Q. 2 & 3: $S = (1000/CN) - 10; I_a = 0.2S$
Runoff - Midsize Catchments: SCS TR55 Method

Peak Discharge, Qp computation:

• Peak Flow (cfs): \( Q_p = q_u A Q F \)

where:

- \( q_u \) = unit peak discharge (cfs/sq. mi/in)
- \( A \) = watershed size in sq. miles
- \( Q \) = Volume of direct runoff in inches
- \( F \) = Pond Factor (depends on % natural storage in ponds and lakes. Assume 1.0 if storage negligible)

• Requires:
  
  • Unit peak discharge, \( q_u \), based on Graphical Method
  
  • Time of Concentration, \( t_c = (\sum \text{Overland} + \sum \text{Channel Flow}) \)
Runoff - Midsize Catchments: SCS TR55 Method
Computing Catchment’s Time of Concentration

SCS TR55 uses the following flow paths for computing catchment's time of concentration:

1. Overland Sheet Flow (<300 feet)
2. Overland Shallow Concentrated Flow
3. Channel or Pipe Flow
SCS TR55 Method (cont.)

1) Equations for Computing Overland Sheet Flow Time:

US Units: \[ t = \frac{0.007(nL)^{0.8}}{P_2^{0.5}S^{0.4}} \]

SI Units: \[ t = \frac{0.02887(nL)^{0.8}}{P_2^{0.5}S^{0.4}} \]

where:
- \( t \) = travel time in hours;
- \( S \) = average land slope in feet/foot (or meters/meter in SI);
- \( n \) = Manning's overland roughness coefficient (see Slide 48);
- \( L \) = overland flow distance in feet (or meters for SI units);
- \( P_2 \) = 2-Year, 24-hour rainfall depth in inches (or cms for SI units).
## Table: TR55 Manning n Values for Overland Sheet Flow

<table>
<thead>
<tr>
<th>Surface Description</th>
<th>Manning n</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth surfaces</td>
<td>0.011</td>
</tr>
<tr>
<td>(concrete, asphalt, gravel, or bare soil)</td>
<td></td>
</tr>
<tr>
<td>Fallow (no residue)</td>
<td>0.05</td>
</tr>
<tr>
<td>Cultivated ground</td>
<td></td>
</tr>
<tr>
<td>(residue cover less than or equal to 20%)</td>
<td>0.06</td>
</tr>
<tr>
<td>(residue cover greater than 20%)</td>
<td>0.17</td>
</tr>
<tr>
<td>Grass</td>
<td></td>
</tr>
<tr>
<td>Range, short prairie</td>
<td>0.15</td>
</tr>
<tr>
<td>Dense</td>
<td>0.24</td>
</tr>
<tr>
<td>Bermuda</td>
<td>0.43</td>
</tr>
<tr>
<td>Range</td>
<td>0.13</td>
</tr>
<tr>
<td>Woods</td>
<td></td>
</tr>
<tr>
<td>Light underbrush</td>
<td>0.40</td>
</tr>
<tr>
<td>Dense underbrush</td>
<td>0.80</td>
</tr>
</tbody>
</table>

*Note: Dense grass includes weeping lovegrass, bluegrass, buffalo grass, blue gamma grass, native grass mixture, alfalfa, and the like.*
SCS TR55 Method (cont.)

2) Computing Shallow Concentrated Overland Flow Time

Shallow Concentrated Overland Flow time is calculated as:

1. Given overland flow path slope use Figure to get overland flow velocity, \( v \) given the overland flow surface as paved or unpaved.

2. Given the length \( L \) of the overland flow path Travel time =

\[
t_2 = \frac{L}{V}
\]
SCS TR55 Method (cont.)

3) Computing Average Velocity for Channel or Pipe Flow Time

Procedure:

**Step 1:** Use Manning’s Formula to compute velocity in pipe or channel:

\[
V = \left(\frac{1.49}{n}\right) R^{2/3} S_0^{1/2} \quad \text{(US Units)}
\]

\[
V = \left(\frac{1.0}{n}\right) R^{2/3} S_0^{1/2} \quad \text{(SI Units)}
\]

(Note: For full pipe flow condition \( R = d_0/4 \) where \( d_0 \) pipe diameter)

**Step 2:** Compute travel time = Pipe or channel length / velocity = \( L/V \)
Example 8: TR-55 Time of Concentration Computation

A 300 acre watershed drains along the path ED→DC→CB→BA shown in the table below. Determine the time of concentration, $t_c$ using SCS TR55 method.

<table>
<thead>
<tr>
<th>Hydraulic Path</th>
<th>Type of Flow</th>
<th>Slope (%)</th>
<th>Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ED</td>
<td>Overland Sheet Flow</td>
<td>5.0</td>
<td>100</td>
</tr>
<tr>
<td>DC</td>
<td>Overland Gutter Flow (unpaved)</td>
<td>1.5</td>
<td>300</td>
</tr>
<tr>
<td>CB</td>
<td>Pipe Flow ($d_0 = 24$ in; $n = 0.015$)</td>
<td>1.0</td>
<td>3000</td>
</tr>
<tr>
<td>BA</td>
<td>Open Channel Flow ($y = 2$ ft; $n = 0.02$)</td>
<td>0.5</td>
<td>5000</td>
</tr>
</tbody>
</table>

(For a wide rectangular: Hydraulic Radius $R =$ flow depth, $y$)

Note: The pipe is 24 inches in diameter with a Manning’s $n = 0.015$.
The open channel is wide rectangular with main bank flow depth = 2.0 ft. and Manning’s $n = 0.020$. 
Example 8: TR-55 Time of Concentration Computation (cont.)

1. **Path ED (Overland Sheet flow):**

   Bermuda Grass – roughness coefficient from slide 48, \( n = 0.43 \)

   2-yr, 24 hour cumulative rainfall depth, \( P_2 = 3.2 \) inches

   Overland surface slope, \( S = 0.05 \)

   Overland flow length, \( L = 100 \) feet

   From Equation – Slide 47

   \[
   \text{Travel time, } t_{ED} = \frac{0.007(nL)^{0.8}}{P_2^{0.5}S^{0.4}} = \frac{0.007(0.43\times100)^{0.8}}{(3.2)^{0.5}(0.05)^{0.4}}
   \]

   \[
   = 0.263 \text{ hrs} = 15.8 \text{ min} = 946.3 \text{ s}
   \]

2. **Path DC (Overland Concentrated Flow – Unpaved Surface):**

   Overland flow slope, \( S = 0.015 \)

   Overland flow velocity, \( V = 1.8 \) ft/s

   From Figure – Slide 49

   \[
   \text{Travel time, } t_{DC} = \frac{L}{V} = \frac{300}{1.8} = 166.67 \text{ s} = 2.78 \text{ min} = 0.046 \text{ hrs}
   \]
3. **Path CB (Pipe Flow):**

- Mannings, $n = 0.015$;
- Pipe slope, $S = 0.01$;
- Pipe length, $L = 3000$ ft
- Pipe diameter = 24 inches = 2.0 feet
- Pipe cross sectional area, $A = 3.14x2^2/4 = 3.14$ ft$^2$
- Pipe wetted perimeter, $P = 3.14x2 = 6.28$ ft
- Pipe Hydraulic Radius, $R = A/P = 3.14/6.28 = 0.50$
- Velocity, $V = (1.49/0.015)x0.5^{2/3}x0.01^{1/2} = 6.26$ ft/s

**Travel time in pipe**, $t_{CB} = L/V = 3000/6.26 = 479.2$ s

$= 7.987$ min $= 0.113$ hrs
Example 8: TR-55 Time of Concentration Computation (cont.)

4. Path BA (Open Channel Flow):

Mannings, n = 0.02; Channel slope, S = 0.005;
Channel length, L = 5000 ft
Flow depth, y = 2.0 feet
Hydraulic Radius, R = y = 2 feet (wide rect. channel)
Velocity, V = \((1.49/0.02)x20^{2/3}x0.005^{1/2}\) = 8.36 ft/s

Travel time in pipe, \(t_{CB} = \frac{L}{V} = \frac{5000}{8.36}\)
= 598.1 s = 9.968 min = 0.166 hrs

Watershed Time of Concentration, \(t_c = \text{total travel time}\)

\[
= \sum t = t_{ED} + t_{DC} + t_{CB} + t_{BA}
\]

= 0.263 + 0.046 + 0.113 + 0.166 = 0.588 hours = 35.28 min
Problem 5: SCS TR-55 - Time of Concentration Computation

Using SCS TR55 Method, calculate the time of concentration of a watershed given the following flow path:

1) **overland sheet flow** on dense grass, length \( L = 100 \text{ ft} \); slope \( S = 0.01 \), 2-yr 24-hr rainfall \( P_2 = 3.6 \text{ inches} \);

2) **shallow concentrated flow** on unpaved surface, length \( L = 1400 \text{ ft} \), slope \( S = 0.01 \);

3) **streamflow**, Manning’s \( n = 0.05 \); flow area \( A = 27 \text{ ft}^2 \), wetted perimeter \( P = 28.2 \text{ ft} \), slope \( S = 0.005 \) and length \( L = 7300 \text{ ft} \).
Problem 5: TR-55 Time of Concentration Computation (cont.)

1) Overland Flow Time, \( t_1 \) (refer to slides 47 and 48):
\[
n = 0.24 \text{ (dense grass)}; \quad L = 100 \text{ ft}; \quad S = 0.01 \text{ and } P_2 = 3.6 \text{ in}
\]
\[
t_1 = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}(S)^{0.4}} = \ldots \text{ min} =
\]

2) Shallow Concentrated Flow, \( t_2 \) (use Figure – Slide 49):
Unpaved; \( L = 1400 \text{ ft}; \quad S = 0.01 \)
Average Velocity = _____ \text{ ft/s}
Travel time, \( t_2 = \frac{L}{V} = \ldots \text{ sec} = \ldots \text{ min} \)

3) Stream/ Channel Flow (use Manning’s equation – Slide 50):
manning’s \( n = 0.05; \quad A = 27 \text{ ft}^2; \quad P = 28.2 \text{ ft}; \quad S = 0.005; \quad L = 7300 \text{ ft} \)
\[
V = \frac{(1.49/n)R^{2/3}S^{1/2}}{} = \ldots \text{ ft/s}
\]
Travel time \( t_3 = \frac{L}{V} = \ldots \text{ sec} = \ldots \text{ min} \)

Total Travel Time (or time of concentration)
\[
= t_1 + t_2 + t_3 = t_c = \ldots \text{min} \ldots \text{ hrs.}
\]
SCS TR55 Method – Peak Discharge Computation

**Steps:**

1. Compute watershed composite curve number CN;
2. Compute sorptivity, $S$ (inches), initial abstraction, $I_a$ (inches) and $I_a/P$ ratio;
3. Compute direct runoff volume $Q$ (inches);
4. Compute unit peak discharge, $q_u$ (cfs/sq. mi/inch) given time of concentration, $t_c$ and $I_a/P$ ratio from Figure in Slide 58 (or similar curve for Type I and III);
5. Determine the pond factor, $F$.
6. Compute peak discharge $q_p = q_u Q A F$
SCS TR55 Method – Peak Discharge Computation

Figure: Unit Peak Discharge Curves-Type II

Refer to USDA TR55 Manual for Type I-A, Type I-B and Type III Curves
Example 9: TR-55 Computation of Peak Flow Using SCSTR55 Graphical Peak Discharge Method

**Given:**

- 250 acre (0.39 sq. miles) watershed;
- 25-year, 24 hour Type II design rainfall $P = 6$ inches;
- watershed time of concentration, $t_c = 1.50$ hours;
- composite SCS Curve number $CN = 75$
- Neglect storage in lakes and ponds.

Compute the peak discharge $q_p$. 
Example 9: TR-55 Computation of Peak Flow
Using SCSTR55 Graphical Peak Discharge Method (Cont..)

Solution:

• $S = \frac{1000}{CN} - 10 = \frac{1000}{75} - 10 = 3.33$ inches;
• $I_a = 0.2 \times S = 0.2 \times 3.33 = 0.667$ inches;
• Ratio $I_a/P = \frac{0.667}{6.0} = 0.11$;
• Runoff volume $Q = \frac{(P-I_a)^2}{(P-I_a+S)}$
  \[= \frac{(6.0-0.667)^2}{(6.0-0.667+3.33)} = 3.28\] inches;
• Unit peak discharge, $q_u = 285$ cfs/sq.mi/inch (Slide 58);
• Pond factor $F = 1$;

Peak Discharge, $q_p = q_u \times Q \times A \times F$
\[= 285 \times 3.28 \times 0.39 \times 1 = 364.6\] cfs.
Runoff Midsize Catchments
Unit Hydrograph Method

- **Unit Hydrograph Definition:**
  The unit hydrograph of a watershed is defined as a direct runoff hydrograph (DRH) resulting from 1 inch (or 1 cm in SI units) of excess rainfall generated uniformly over the drainage area at a constant rate for an effective duration $t_r$ (referred to as the Unit Hydrograph duration)

(Adapted from Ponce 1989)
Unit Hydrograph Properties
(Adapted from Ponce 1989)

Linearity

Superposition

Lagging
Direct Runoff Convolution (Adapted from Chow et al (1988))

The set of equations for discrete time convolution

\[ Q_n = \sum_{m=1}^{n \leq M} P_m U_{n-m+1} ; \]

\[ n = 1, 2, \ldots, N \]

\[ Q_1 = P_1 U_1 \]
\[ Q_2 = P_2 U_1 + P_1 U_2 \]
\[ Q_3 = P_3 U_1 + P_2 U_2 + P_1 U_3 \]
\[ \ldots \]
\[ Q_M = P_M U_1 + \ldots + P_{M-1} U_2 + P_1 U_M \]
\[ Q_{M-1} = 0 + \ldots + P_2 U_{M-1} + P_1 U_{M+1} \]
\[ \ldots \]
\[ Q_{N-1} = 0 + 0 + \ldots + 0 + 0 + \ldots + P_M U_{N-M} + P_{M-1} U_{N-M+1} \]
\[ Q_N = 0 + 0 + \ldots + 0 + 0 + \ldots + 0 + P_M U_{N-M+1} \]

In the equations above:

- \( P_m \) = rainfall excess pulses (inches or cms)
- \( U_j \) = unit hydrograph ordinates cfs/in or m³/s/cms
- \( Q_n \) = Direct runoff ordinates (cfs)
- \( N \) = number of non-zero direct runoff ordinates, \( Q_n \)
- \( M \) = number of rainfall excess pulses in the hyetograph, \( P_m \)
- \( J \) = number of Unit Hydrograph ordinates, \( U_j \)

\[ N = M + J - 1 \]
Example 10: Derivation of Direct Runoff and Streamflow Hydrographs Using a Given Unit Hydrograph (Chow et al 1988)

Given:

• ½ hour unit hydrograph
• Storm of 6 inches total rainfall excess depth with:
  2 inches first half hour ($P_1$),
  3 inches in the second half hour ($P_2$)
  1 inch in the third half hour ($P_3$)
• Base Flow = 500 cfs
• Determine:
  Direct Runoff and the Streamflow Hydrograph.
Example 10: Derivation of Direct Runoff and Stream Flow Hydrographs Using a Given Unit Hydrograph (cont.)

Solution by Convolution where $N = M + J - 1 = 3 + 9 - 1 = 11$ ordinates

<table>
<thead>
<tr>
<th>Time Interval $\Delta t$ = ½ hr</th>
<th>J = 9</th>
<th>½ Hour Unit Hydrograph</th>
<th>Includes base flow = 500 cfs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excess Precipitation (in)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Time (t-h)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$n$</th>
<th>Precipitation (in)</th>
<th>$P's$</th>
<th>$U's$</th>
<th>$Q's$</th>
<th>Direct runoff (cfs)</th>
<th>Streamflow* (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.00</td>
<td>808</td>
<td></td>
<td></td>
<td>808</td>
<td>1308</td>
</tr>
<tr>
<td>2</td>
<td>3.00</td>
<td>1212</td>
<td>2158</td>
<td></td>
<td>3370</td>
<td>3870</td>
</tr>
<tr>
<td>3</td>
<td>1.00</td>
<td>404</td>
<td>3237</td>
<td>4686</td>
<td>8327</td>
<td>8827</td>
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<tr>
<td>4</td>
<td></td>
<td>1079</td>
<td>7029</td>
<td>5012</td>
<td>13,120</td>
<td>13,620</td>
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<td>5</td>
<td></td>
<td>2343</td>
<td>7518</td>
<td>2920</td>
<td>12,781</td>
<td>13,281</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>2506</td>
<td>4380</td>
<td>906</td>
<td>7792</td>
<td>8292</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>1460</td>
<td>1359</td>
<td>762</td>
<td>3581</td>
<td>4081</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>1453</td>
<td>1143</td>
<td>548</td>
<td>2144</td>
<td>2644</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>381</td>
<td>822</td>
<td>346</td>
<td>1549</td>
<td>2049</td>
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<tr>
<td>10</td>
<td></td>
<td>274</td>
<td>519</td>
<td>793</td>
<td>1293</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
<td>173</td>
<td>173</td>
<td>673</td>
<td>Total</td>
<td>54,438</td>
</tr>
</tbody>
</table>

Baseflow = 500 cfs.
Problem 6: Derivation of Direct Runoff and Streamflow Hydrographs Using a Given Unit Hydrograph

Given: Rainfall hyetograph, one-hour unit hydrograph and abstractions a constant rate of $\Phi = 0.5$ in/hr.

<table>
<thead>
<tr>
<th>Time (hrs):</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall intensity, i (in/hr):</td>
<td>1.0</td>
<td>1.5</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>1-hr Unit Hydrograph (cfs/in):</td>
<td>50</td>
<td>200</td>
<td>150</td>
<td>50</td>
</tr>
</tbody>
</table>

Answer the following questions by completing the spaces shown in green:

a) Time (hrs):
   - Rainfall Excess (inches): [___] [___] [___]

b) Time (hrs):
   - Direct Runoff, Q (cfs): [___] [___] [___] [___] [___]
   - (see slide 65)

c) Volume of Direct Runoff in cfs-hours = [___]

d) Watershed size in sq. mile = [___]
Thank you for listening to the presentation.

Remember to Review the following Appendix for:

1. Hydrologic Routing
2. Groundwater Hydrology – Well Hydraulics
3. Additional examples on Unit Hydrograph

Good luck on the P.E. Exam

QED
REFERENCES


Answers

- Problem 1 (Slide 9): 1) a;  2) a.
- Problem 1 (Slide 10): 3) b;  4) c;  5) b;  6) c.
- Problem 2 (Slide 21): 1) b;  2) d;  3) c.
- Problem 3 (Slide 34): 1) c;  2) b;  3) d;  4) d.
- Problem 4 (Slide 44): 1) c;  2) c;  3) d.
- Problem 5 (Slide 55): 1) Overland sheet flow: 0.296 hrs;
  2) Overland Shallow Concentrated flow: 0.229 hrs
  3) Stream Flow: 0.991 hrs
  4) Total Time of Concentration = 1.515 hours.
- Problem 6 (Slide 66): a) Time (hrs):
  1  2  3
  Rainfall excess (inches): 0.5  1.0  0  (M=2)
b) Time (hrs):
  1  2  3  4  5
  Direct runoff (cfs): 25  150  275  175  50  (N=5)
c) Volume of direct runoff, \( V_d = 675 \text{ cfs-hours} = 2430,000 \text{ ft}^3 \)
d) Drainage Area = 0.697 sq. miles
- (Hint: use volume under unit hydrograph = 1 inch or Volume under direct runoff hydrograph)
APPENDIX A

Hydrologic Design Components for SCS TR55
Table: Design Frequency or Return Period for Various Hydraulic Structures

<table>
<thead>
<tr>
<th>Type of structure</th>
<th>Return period (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highway culverts</td>
<td></td>
</tr>
<tr>
<td>Low traffic</td>
<td>5–10</td>
</tr>
<tr>
<td>Intermediate traffic</td>
<td>10–25</td>
</tr>
<tr>
<td>High traffic</td>
<td>50–100</td>
</tr>
<tr>
<td>Highway bridges</td>
<td></td>
</tr>
<tr>
<td>Secondary system</td>
<td>10–50</td>
</tr>
<tr>
<td>Primary system</td>
<td>50–100</td>
</tr>
<tr>
<td>Farm drainage</td>
<td></td>
</tr>
<tr>
<td>Culverts</td>
<td>5–50</td>
</tr>
<tr>
<td>Ditches</td>
<td>5–50</td>
</tr>
<tr>
<td>Urban drainage</td>
<td></td>
</tr>
<tr>
<td>Storm sewers in small cities</td>
<td>2–25</td>
</tr>
<tr>
<td>Storm sewers in large cities</td>
<td>25–50</td>
</tr>
<tr>
<td>Airfields</td>
<td></td>
</tr>
<tr>
<td>Low traffic</td>
<td>5–10</td>
</tr>
<tr>
<td>Intermediate traffic</td>
<td>10–25</td>
</tr>
<tr>
<td>High traffic</td>
<td>50–100</td>
</tr>
<tr>
<td>Levees</td>
<td></td>
</tr>
<tr>
<td>On farms</td>
<td>2–50</td>
</tr>
<tr>
<td>Around cities</td>
<td>50–200</td>
</tr>
<tr>
<td>Dams with no likelihood of loss of life (low hazard)</td>
<td></td>
</tr>
<tr>
<td>Small dams</td>
<td>50–100</td>
</tr>
<tr>
<td>Intermediate dams</td>
<td>100 +</td>
</tr>
<tr>
<td>Large dams</td>
<td>—</td>
</tr>
<tr>
<td>Dams with probable loss of life (significant hazard)</td>
<td></td>
</tr>
<tr>
<td>Small dams</td>
<td>100 +</td>
</tr>
<tr>
<td>Intermediate dams</td>
<td>—</td>
</tr>
<tr>
<td>Large dams</td>
<td>—</td>
</tr>
<tr>
<td>Dams with high likelihood of considerable loss of life (high hazard)</td>
<td></td>
</tr>
<tr>
<td>Small dams</td>
<td>—</td>
</tr>
<tr>
<td>Intermediate dams</td>
<td>—</td>
</tr>
<tr>
<td>Large dams</td>
<td>—</td>
</tr>
</tbody>
</table>
Figure: Rainfall Depth & Intensity

Louisville, KY
100-yr, 6-hr Rainfall
= 4.5 inches

100-year 6-hr rainfall (inches). (U.S. Weather Bureau [38].)
adapted from Chow, Handbook of Applied Hydrology, McGraw – Hill 1964
Methods for Computing Time of Concentration

• Formulas
  – Example: Kirpich, SCS Average Velocity Charts

• Approximate Velocities
Table: Formulas for Time of Concentration, $t_c$
(source: Chow et al, 1988)

Summary of time of concentration formulas

<table>
<thead>
<tr>
<th>Method and Date</th>
<th>Formula for $t_c$ (min)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kirpich (1940)</td>
<td>$t_c = 0.0078 L^{0.77} S^{-0.385}$</td>
<td>Developed from SCS data for seven rural basins in Tennessee with well-defined channel and steep slopes (3% to 10%); for overland flow on concrete or asphalt surfaces multiply $t_c$ by 0.4; for concrete channels multiply by 0.2; no adjustments for overland flow on bare soil or flow in roadside ditches.</td>
</tr>
<tr>
<td></td>
<td>$L =$ length of channel/ditch from headwater to outlet, ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$S =$ average watershed slope, ft/ft</td>
<td></td>
</tr>
<tr>
<td>California Culverts Practice</td>
<td>$t_c = 60(11.9 L^3/H)^0.385$</td>
<td>Essentially the Kirpich formula; developed from small mountainous basins in California (U. S. Bureau of Reclamation, Source: Kibler, 1982, Copyright by the American Geophysical Union.</td>
</tr>
<tr>
<td>(1942)</td>
<td>$L =$ length of longest</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$H =$ elevation difference between divide and outlet, ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Source: Kibler, 1982, Copyright by the American Geophysical Union.</td>
<td></td>
</tr>
<tr>
<td>Izzard (1946)</td>
<td>$t_c = \frac{41.025(0.0007i + c)L^{0.33}}{S^{0.332}i^{0.667}}$</td>
<td>Developed in laboratory experiments by Bureau of Public Roads for overland flow on roadway and turf surfaces; values of the retardance coefficient range from 0.0070 for very smooth pavement to 0.012 for concrete pavement to 0.06 for dense turf; solution requires iteration; product $i$ times $L$ should be $\leq 500$.</td>
</tr>
<tr>
<td></td>
<td>$i =$ rainfall intensity, in/h</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$c =$ retardance coefficient</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$L =$ length of flow path, ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$S =$ slope of flow path, ft/ft</td>
<td></td>
</tr>
<tr>
<td>Federal Aviation Administration</td>
<td>$t_c = 1.8(1.1 - C)L^{0.50}/S^{0.333}$</td>
<td>Developed from air field drainage data assembled by the Corps of Engineers; method is intended for use on airfield drainage problems, but has been used frequently for overland flow in urban basins.</td>
</tr>
<tr>
<td>(1970)</td>
<td>$C =$ rational method runoff coefficient</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$L =$ length of overland flow, ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$S =$ surface slope, %</td>
<td></td>
</tr>
</tbody>
</table>

Source: Kibler, 1982, Copyright by the American Geophysical Union.
### Summary of time of concentration formulas

<table>
<thead>
<tr>
<th>Method and Date</th>
<th>Formula for $t_c$ (min)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kinematic wave formulas</td>
<td>$t_c = \frac{0.94 L^{0.6} n^{0.6}}{(i^{0.45} S^{0.3})}$</td>
<td>Overland flow equation developed from kinematic wave analysis of surface runoff from developed surfaces; method requires iteration since both $i$ (rainfall intensity) and $t_c$ are unknown; superposition of intensity–duration–frequency curve gives direct graphical solution for $t_c$</td>
</tr>
<tr>
<td>Morgali and Linsley (1965)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aron and Erborge (1973)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCS lag equation (1973)</td>
<td>$t_c = \frac{100 L^{0.8}((1000/CN) - 9)^{0.7}}{1900 S^{0.5}}$</td>
<td>Equation developed by SCS from agricultural watershed data; it has been adapted to small urban basins under 2000 acres; found generally good where area is completely paved; for mixed areas it tends to underestimate; adjustment factors are applied to correct for channel improvement and impervious area; the equation assumes that $t_c = 1.67 \times$ basin lag.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCS average velocity charts</td>
<td>$t_c = \frac{L}{60 - V}$</td>
<td>Overland flow charts in Fig. 3–1 of TR 55 show average velocity as function of watercourse slope and surface cover. (See also Table 5.7.1)</td>
</tr>
<tr>
<td>(1975, 1986)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Source: Kibler, 1982. Copyright by the American Geophysical Union.*
Table: Average Velocities for Different Flow Paths

*(source: Chow et al. 1988)*

<table>
<thead>
<tr>
<th>Description of water course</th>
<th>Slope in percent</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0–3</td>
</tr>
<tr>
<td>Unconcentrated*</td>
<td></td>
</tr>
<tr>
<td>Woodlands</td>
<td>0–1.5</td>
</tr>
<tr>
<td>Pastures</td>
<td>0–2.5</td>
</tr>
<tr>
<td>Cultivated</td>
<td>0–3.0</td>
</tr>
<tr>
<td>Pavements</td>
<td>0–8.5</td>
</tr>
<tr>
<td>Concentrated**</td>
<td></td>
</tr>
<tr>
<td>Outlet channel—determine velocity by Manning’s formula</td>
<td></td>
</tr>
<tr>
<td>Natural channel not well defined</td>
<td>0–2</td>
</tr>
</tbody>
</table>

*This condition usually occurs in the upper extremities of a watershed prior to the overland flows accumulating in a channel.*

**These values vary with the channel size and other conditions. Where possible, more accurate determination should be made for particular conditions by the Manning channel formula for velocity.

*(Source: Drainage Manual, Texas Highway Department, Table VII, p. II-28, 1970.)*
APPENDIX B

Hydrologic Routing
Reservoir or Detention Basin Routing

- **Storage – Indication Method:** Used for routing flood hydrographs through detention basins or reservoirs.

- **Information Required:**
  
  1. **Storage-Elevation Data** for the reservoir (obtained from site topographic map)
  
  2. **Storage-Discharge Relationship** (depends on the hydraulics of outflow control structures such as a spillway) (See Figure – Slide 67)
  
  3. Steps 1 and 2 are combined to develop a Storage-Indication curve:
     \[
     \frac{2S}{\Delta t} + Q \text{ vs } Q \text{ curve (See Figure – Slide 68)}
     \]

- **Uses discrete form of equation of continuity as:**

\[
\left(\frac{2S_{j+1}}{\Delta t} - Q_{j+1}\right) = \left(\frac{2S_{j+1}}{\Delta t} + Q_{j+1}\right) - 2Q_{j+1}
\]
Reservoir Routing (cont.)

Figure: Discharge-Elevation Relationships for Various Types of Spillway Structures
(Adapted from: Chow et al, 1988)

<table>
<thead>
<tr>
<th>Spillway type</th>
<th>Equation</th>
<th>Notation</th>
</tr>
</thead>
</table>
| Uncontrolled overflow ogee crest     | $Q = CLH^{3/2}$                 | $Q =$ discharge, cfs  
|                                      |                                 | $C =$ variable coefficient of discharge  
|                                      |                                 | $L =$ effective length of crest  
|                                      |                                 | $H =$ total head on the crest including velocity of approach head. |
| Gate controlled ogee crest           | $Q = \frac{2}{3} \sqrt{2gCL(H_1^{3/2} - H_2^{3/2})}$ | $H_1 =$ total head to bottom of the opening  
|                                      |                                 | $H_2 =$ total head to top of the opening  
|                                      |                                 | $C =$ coefficient which differs with gate and crest arrangement |
| Morning glory spillway               | $Q = C_o(2\pi R_s)H^{3/2}$     | $C_o =$ coefficient related to $H$ and $R_s$  
|                                      |                                 | $R_s =$ radius of the overflow crest  
|                                      |                                 | $H =$ total head |
| Culvert (submerged inlet control)    | $Q = C_d WD \sqrt{2gH}$        | $W =$ entrance width  
|                                      |                                 | $D =$ height of opening  
|                                      |                                 | $C_d =$ discharge coefficient |

Reservoir Routing (cont.)
Figure: Development of Storage Indication Curve
(Chow et al, 1988)
Reservoir Routing (cont.)
Example 8: Developing a Storage Indication Curve.

Problem Statement:
Storage vs outflow characteristics for a proposed reservoir are given below. Calculate the storage-outflow function $2S/\Delta t + Q$ vs $Q$ for each of the tabulated values if $\Delta t = 2$ hours. Plot a graph of the storage-outflow function.

<table>
<thead>
<tr>
<th>Storage, $S$ ($10^6$ m$^3$):</th>
<th>75</th>
<th>81</th>
<th>87.5</th>
<th>100</th>
<th>110.2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outflow, $Q$ (m$^3$/s):</td>
<td>57</td>
<td>227</td>
<td>519</td>
<td>1330</td>
<td>2270</td>
</tr>
</tbody>
</table>

Solution:
Reservoir Routing
Example 8: Developing a Storage Indication Curve (cont.)
(Adapted from: Chow et al, 1988)

Storage-Outflow, S vs Q

Storage Indication Curve, 2S/Δt + Q vs Q
Example 9: Reservoir Routing (Cont.)
(Chow et al, 1988)

Using the following storage indication data route the given inflow hydrograph using the storage indication method.

<table>
<thead>
<tr>
<th>Column:</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time</td>
<td>Elevation $H$ (ft)</td>
<td>Discharge $Q$ (cfs)</td>
<td>Storage $S$ (ft$^3$)</td>
</tr>
<tr>
<td>index $j$</td>
<td>Time (min)</td>
<td>Inflow (cfs)</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0.5</td>
<td>3</td>
<td>21.780</td>
</tr>
<tr>
<td>3</td>
<td>1.0</td>
<td>8</td>
<td>43.560</td>
</tr>
<tr>
<td>4</td>
<td>1.5</td>
<td>17</td>
<td>65.340</td>
</tr>
<tr>
<td>5</td>
<td>2.0</td>
<td>30</td>
<td>87.120</td>
</tr>
<tr>
<td>6</td>
<td>2.5</td>
<td>43</td>
<td>108.900</td>
</tr>
<tr>
<td>7</td>
<td>3.0</td>
<td>60</td>
<td>130.680</td>
</tr>
<tr>
<td>8</td>
<td>3.5</td>
<td>78</td>
<td>152.460</td>
</tr>
<tr>
<td>9</td>
<td>4.0</td>
<td>97</td>
<td>174.240</td>
</tr>
<tr>
<td>10</td>
<td>4.5</td>
<td>117</td>
<td>196.020</td>
</tr>
<tr>
<td>11</td>
<td>5.0</td>
<td>137</td>
<td>217.800</td>
</tr>
<tr>
<td>12</td>
<td>5.5</td>
<td>156</td>
<td>239.580</td>
</tr>
<tr>
<td>13</td>
<td>6.0</td>
<td>173</td>
<td>261.360</td>
</tr>
<tr>
<td>14</td>
<td>6.5</td>
<td>190</td>
<td>283.140</td>
</tr>
<tr>
<td>15</td>
<td>7.0</td>
<td>205</td>
<td>304.920</td>
</tr>
<tr>
<td>16</td>
<td>7.5</td>
<td>218</td>
<td>326.700</td>
</tr>
<tr>
<td>17</td>
<td>8.0</td>
<td>231</td>
<td>348.480</td>
</tr>
<tr>
<td>18</td>
<td>8.5</td>
<td>242</td>
<td>370.260</td>
</tr>
<tr>
<td>19</td>
<td>9.0</td>
<td>253</td>
<td>392.040</td>
</tr>
<tr>
<td>20</td>
<td>9.5</td>
<td>264</td>
<td>413.820</td>
</tr>
<tr>
<td>21</td>
<td>10.0</td>
<td>275</td>
<td>435.600</td>
</tr>
</tbody>
</table>

*Time interval $\Delta t = 10$ min.
Example 9 Reservoir Routing (Chow et al, 1988) (cont.):

Solution:

From Storage Indication Equation

\[
\frac{2S_{j+1}}{\Delta t} + Q_{j+1} = (I_{j+1} + I_j) + \left(\frac{2S_j}{\Delta t} - Q_j\right)
\]

From Storage Indication Curve Table Slide 73

Solve this equation for next time step \( j = 3 \)

<table>
<thead>
<tr>
<th>Column:</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time</td>
<td>Time</td>
<td>Inflow</td>
<td>( I_j )</td>
<td>( I_{j-1} )</td>
<td>( \frac{2S_j}{\Delta t} )</td>
<td>( Q_j )</td>
</tr>
<tr>
<td>index ( j )</td>
<td>(min)</td>
<td>(cfs)</td>
<td>(cfs)</td>
<td>(cfs)</td>
<td>(cfs)</td>
<td>(cfs)</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>60</td>
<td>60</td>
<td>120</td>
<td>55.2</td>
<td>60.0</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
<td>120</td>
<td>180</td>
<td>300</td>
<td>201.1</td>
<td>235.2</td>
</tr>
<tr>
<td>4</td>
<td>30</td>
<td>180</td>
<td>300</td>
<td>600</td>
<td>378.9</td>
<td>501.1</td>
</tr>
<tr>
<td>5</td>
<td>40</td>
<td>240</td>
<td>420</td>
<td>920</td>
<td>552.0</td>
<td>798.9</td>
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<td>6</td>
<td>50</td>
<td>300</td>
<td>540</td>
<td>1260</td>
<td>728.2</td>
<td>1092.6</td>
</tr>
<tr>
<td>7</td>
<td>60</td>
<td>360</td>
<td>660</td>
<td>1620</td>
<td>927.5</td>
<td>1388.2</td>
</tr>
<tr>
<td>8</td>
<td>70</td>
<td>420</td>
<td>780</td>
<td>2060</td>
<td>1089.0</td>
<td>1607.5</td>
</tr>
<tr>
<td>9</td>
<td>80</td>
<td>480</td>
<td>900</td>
<td>2500</td>
<td>1149.0</td>
<td>1899.0</td>
</tr>
<tr>
<td>10</td>
<td>90</td>
<td>520</td>
<td>1020</td>
<td>3000</td>
<td>1134.3</td>
<td>1669.0</td>
</tr>
<tr>
<td>11</td>
<td>100</td>
<td>580</td>
<td>1140</td>
<td>3500</td>
<td>1064.4</td>
<td>1574.3</td>
</tr>
<tr>
<td>12</td>
<td>110</td>
<td>640</td>
<td>1260</td>
<td>4000</td>
<td>954.1</td>
<td>1424.4</td>
</tr>
<tr>
<td>13</td>
<td>120</td>
<td>700</td>
<td>1380</td>
<td>4500</td>
<td>820.2</td>
<td>1234.1</td>
</tr>
<tr>
<td>14</td>
<td>130</td>
<td>760</td>
<td>1500</td>
<td>5000</td>
<td>683.3</td>
<td>1020.2</td>
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<tr>
<td>15</td>
<td>140</td>
<td>820</td>
<td>1620</td>
<td>5500</td>
<td>555.1</td>
<td>803.3</td>
</tr>
<tr>
<td>16</td>
<td>150</td>
<td>880</td>
<td>1740</td>
<td>6000</td>
<td>435.4</td>
<td>595.1</td>
</tr>
<tr>
<td>17</td>
<td>160</td>
<td>940</td>
<td>1860</td>
<td>6500</td>
<td>338.2</td>
<td>435.4</td>
</tr>
<tr>
<td>18</td>
<td>170</td>
<td>1000</td>
<td>1980</td>
<td>7000</td>
<td>272.8</td>
<td>338.2</td>
</tr>
<tr>
<td>19</td>
<td>180</td>
<td>1060</td>
<td>2100</td>
<td>7500</td>
<td>227.3</td>
<td>272.8</td>
</tr>
<tr>
<td>20</td>
<td>190</td>
<td>1120</td>
<td>2220</td>
<td>8000</td>
<td>194.9</td>
<td>227.3</td>
</tr>
<tr>
<td>21</td>
<td>200</td>
<td>1180</td>
<td>2340</td>
<td>8500</td>
<td>169.7</td>
<td>194.9</td>
</tr>
<tr>
<td>22</td>
<td>210</td>
<td>1240</td>
<td>2460</td>
<td>9000</td>
<td>145.7</td>
<td>169.7</td>
</tr>
</tbody>
</table>

Column 5

Column 6

inflow hydrograph

outflow hydrograph
Reservoir Routing
Example: Reservoir Routing Using Storage-Indication Method.
(Adapted from: Chow et al, 1988)

Problem Statement:

Use the level pool routing method to route the hydrograph given below through the reservoir whose storage-outflow characteristics are given in Prob. 8.2.1. What is the maximum reservoir discharge and storage? Assume that the reservoir initially contains $75 \times 10^6$ m$^3$ of storage.

<table>
<thead>
<tr>
<th>Time (h)</th>
<th>0</th>
<th>2</th>
<th>4</th>
<th>6</th>
<th>8</th>
<th>10</th>
<th>12</th>
<th>14</th>
<th>16</th>
<th>18</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inflow (m$^3$/sec)</td>
<td>60</td>
<td>100</td>
<td>232</td>
<td>300</td>
<td>520</td>
<td>1,310</td>
<td>1,930</td>
<td>1,460</td>
<td>930</td>
<td>650</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Storage ($10^6$ m$^3$)</th>
<th>Outflow (m$^3$/s)</th>
<th>$2S/\Delta t + Q$ (m$^3$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>75</td>
<td>57</td>
<td>20890</td>
</tr>
<tr>
<td>81</td>
<td>227</td>
<td>22727</td>
</tr>
<tr>
<td>87.5</td>
<td>519</td>
<td>24825</td>
</tr>
<tr>
<td>100</td>
<td>1330</td>
<td>29108</td>
</tr>
<tr>
<td>110.2</td>
<td>2270</td>
<td>32881</td>
</tr>
</tbody>
</table>

Storage Indication Curve
$(2S/dT + Q)$ vs $Q$
Reservoir Routing (Cont.)
Example: Reservoir Routing Using Storage-Indication Method (cont.).
(Adapted from: Chow et al, 1988)

Solution:

Column:
1. Time (min)
2. Inflow (cfs)
3. \( I_j + I_{j+1} \) (cfs)
4. \( 2S_j/\Delta t - Q_j \) (cfs)
5. \( 2S_{j+1}/\Delta t + Q_{j+1} \) (cfs)
6. Outflow, \( Q_j \) (cfs)

\[
\begin{align*}
\text{Storage Indication Curve:} & \\
\text{Discharge, Q (m3/s):} & \\
\text{Storage Indication:} & \\
2S/dT + Q & \\
\end{align*}
\]
APPENDIX C

Groundwater: Well Hydraulics
GROUND WATER HYDROLOGY

Terminology

• **Aquifer** – water bearing strata capable of transmitting water (fluid) at a rate as to be suitable for water supply

• **Confined Aquifer** – an aquifer located between two layers of low permeability or impermeable strata (aquitard)

• **Unconfined Aquifer** – water table aquifer, phreatic aquifer, water level exposed to atmospheric pressure

• **Vadose zone** – unsaturated region above the water table

• **Potentiometric or Piezometric surface** – hydraulic head, $h$, level at a point or across a region of aquifer
Fundamental Principles

Darcy’s Law: \( q = K i \)

- \( q \equiv \) specific discharge \{L/T\}
- \( K \equiv \) hydraulic conductivity \{L/T\}, (ft/s, m/s)
- \( i \equiv \) hydraulic gradient \( = \frac{dh}{dL} \) \{dimensionless, L/L\}

\( h = \) hydraulic head, \( h \)

\( = \psi + z \)

- \( \psi \equiv \) gage pressure head or water pressure \( = \frac{p_{gage}}{\gamma} \) \{L\}
- \( z \equiv \) elevation head \{L\}

Note: \( \psi = 0 \) in saturated zone
Fundamental Principles (cont.)

Darcy’s Law: \( v = q = K \ i \)

- \( v = q = \) flux rate or velocity (ft/s or m/s)
- \( i = \) hydraulic gradient, \( \frac{dh}{dL} = (h_2 - h_1)/(L_2 - L_1) \)
- \( K = \) hydraulic conductivity (ft/s or m/s)

Volume flow rate: \( Q = q \cdot A \)

- \( A = \) flow area
Well Hydraulics

Thiem’s Steady State Solution

- **Unconfined Aquifer:**
  \[ Q = \frac{\pi K_f (h_2^2 - h_1^2)}{\ln(r_2/r_1)} \]

- **Confined Aquifer:**
  \[ Q = 2\pi K_f m \frac{h_2 - h_1}{\ln(r_2/r_1)} \]

where,
- \( Q \) = Discharge from pumping well in cfs (m³/s or gpm);
- \( h \) = hydraulic or piezometric head = \( p/\gamma \) in ft (or m);
- \( K_f \) = hydraulic conductivity in gpd/ft² (or ft/s);
- \( r \) = distance to observation well from center of the pumping well in ft (or m);
- \( m \) = thickness of the confined aquifer in ft (or m)
Example: Thiem’s Steady State Solution - Unconfined Aquifer

\[ Q = \frac{\pi K_f (h_2^2 - h_1^2)}{\ln(r_2/r_1)} \]
Example: Thiem’s Steady State Solution - Unconfined Aquifer

Given:

• A 20 inch diameter well fully penetrates a 100 ft deep unconfined aquifer;
• Drawdowns at two observation wells located at 90 ft and 240 ft from the pumping well are 23 ft and 21.5 ft respectively;
• Hydraulic Conductivity of the aquifer is $K_f = 1400 \text{ gpd/ft}^2 \times (1.55 \times 10^{-6} \text{ cfs/gpd}) = 2.17 \times 10^{-3} \text{ ft/s}$
• Determine: The discharge $Q$ from the pumping well in gpm

Solution:

Hydraulic heads:
- $h_1 = 100 - 23 = 77 \text{ ft} \quad (\text{at } r_1 = 90 \text{ ft})$;
- $h_2 = 100 - 21.5 = 78.5 \text{ ft} \quad (\text{at } r_2 = 240 \text{ ft})$;
- $K_f = 1400 \text{ gpd/ft}^2 \times (1.55 \times 10^{-6} \text{ cfs/gpd}) = 2.17 \times 10^{-3} \text{ ft/s}$

$Q = \frac{\pi K_f (h_2^2 - h_1^2)}{\ln(r_2/r_1)} - \text{all in consistent units}$

$Q = 3.14 \times 2.17 \times 10^{-3} (78.5^2 - 77^2) / \ln(240/90)$

$= 1.62 \text{ cfs} = 725.9 \text{ gpm}$
Example: Thiem’s Steady State Solution – Confined Aquifer

\[ Q = 2\pi K_f m \frac{h_2 - h_1}{\ln(r_2/r_1)} \]

FIGURE 3.9 Radial flow to a well in a confined aquifer.
Example: Thiem’s Steady State Solution – Confined Aquifer

Problem: Determine the hydraulic conductivity, \( K_f \), of an artesian aquifer (confined aquifer) pumped by a fully penetrating well.

Given:
1) Aquifer thickness, \( m = 100 \) feet
2) Steady state pumping rate \( Q = 1000 \) gpm = 2.232 cfs
3) Drawdowns, \( s \), at observation wells:
   - Well 1: \( r_1 = 50 \) ft; \( s_1 = 10 \) (\( h_1 = 100-10 = 90 \) ft)
   - Well 2: \( r_2 = 500 \) ft; \( s_2 = 1 \) ft (\( h_2 = 100-1 = 99 \) ft)

Solution:
Solve for Hydraulic Conductivity, \( K_f \) from Thiem’s steady state equation for confined aquifer:

\[
K_f = \frac{Q \left( \log_e \left( \frac{r_2}{r_1} \right) \right) }{ \left\{ 2 \pi m \left( h_2 - h_1 \right) \right\} } - \text{all in consistent units}
\]

\[
= \frac{2.232 \times \log_e \left( \frac{500}{50} \right) }{2 \times 3.14 \times 100 \times (99-90)}
\]

\[
= 9.093 \times 10^{-4} \text{ ft/s or ft}^3/\text{s.ft}^2
\]

\[
= (9.093 \times 10^{-4} \text{ft}^3/\text{s.ft}^2) \times 646,323 \text{ gpd/ft}^2
\]

\[
= \text{587.7 gpd/ft}^2
\]
Theis Unsteady State Solution

ASSUMPTIONS
1. The aquifer is homogeneous, isotropic, and of infinite extent (this is a built-in assumption of the groundwater flow equation).
2. The transmissivity of the aquifer is practically constant.
3. The water derived is entirely from storage and is released instantaneously with decline of head.
4. The well penetrates the entire thickness of the aquifer, and its diameter is very small compared to pumping rates, so that storage in well is negligible.

INITIAL AND BOUNDARY CONDITIONS
1. At time = 0, drawdown = 0, at any distance.
2. At time > 0, drawdown = 0, at infinite distance.
Theis Unsteady State Solution - Equations

\[ h_0 - h = s = \frac{Q}{4\pi T} W(u) \]

\[ u = \frac{r^2 S}{4 T t} \]

In US practice – Equations are used with Q in gpm, T in gpd/ft and time t = days

\[ s = \frac{114.6Q}{T} \int_0^\infty \frac{e^{-\frac{u}{u}}}{u} du \]

\[ u = \frac{1.87r^2S}{Tt} \]

Variables:
- Drawdown: \( s = h_0 - h \)
- hydraulic conductivity: \( K \)
- aquifer transmissivity: \( T = K b \)
- aquifer thickness (confined) or saturated thickness (unconfined) : \( b \)
- Storativity: \( S \)
- well function: \( W(u) \)
- \( r = \) radial distance from center of well
Theis Unsteady State Solution – Well Function $W(u)$

Well function, $W(u)$ a function of the term $u$
Tabulated to provide a convenient method to estimate drawdown for given aquifer conditions and steady-state pumping rate, $Q$

<table>
<thead>
<tr>
<th>$u$</th>
<th>1.0</th>
<th>2.0</th>
<th>3.0</th>
<th>4.0</th>
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<th>6.0</th>
<th>7.0</th>
<th>8.0</th>
<th>9.0</th>
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<tbody>
<tr>
<td>$\times 1$</td>
<td>0.219</td>
<td>0.049</td>
<td>0.013</td>
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<td>0.0000012</td>
<td>0.00000012</td>
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<tr>
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<td>0.37</td>
<td>0.31</td>
<td>0.26</td>
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<td>2.30</td>
<td>2.15</td>
<td>2.03</td>
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<td>5.23</td>
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<td>4.73</td>
<td>4.54</td>
<td>4.39</td>
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<td>4.14</td>
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<td>7.02</td>
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<td>9.33</td>
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<td>13.73</td>
<td>13.58</td>
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<td>13.34</td>
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<td>25.26</td>
<td>25.11</td>
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<tr>
<td>$\times 10^{-14}$</td>
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<td>30.56</td>
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<td>29.71</td>
<td>29.58</td>
<td>29.46</td>
</tr>
<tr>
<td>$\times 10^{-15}$</td>
<td>33.96</td>
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<td>32.86</td>
<td>32.58</td>
<td>32.35</td>
<td>32.17</td>
<td>32.02</td>
<td>31.88</td>
<td>31.76</td>
</tr>
</tbody>
</table>

Example: Theis Method

Given:
A well is pumped at \( Q = 5400 \text{ m}^3/\text{day} \)
Aquifer properties:
\( S = 0.0003; \)
\( T = 2200 \text{ m}^2/\text{day} \ (0.0025 \text{ m}^2/\text{s}) \)

Compute:
Drawdown, \( s = h_0 - h \),
at \( t = 10 \) days, \( r = 20 \) m

Solution:
Compute \( u = 1.36.10^{-6} \)
Well Function: \( W(u) = 12.99 \)

\[
h_0 - h = s = \frac{Q}{4\pi T} W(u)
\]

Drawdown, \( s = \{5400/(4\times3.14\times2200)\} \times 12.99 = 2.53 \text{ m} \)
Problem 7: Drawdown by Theis method

Given:

• The following information for a confined aquifer:
  a) Transmissivity \( T = 1650 \text{ ft}^2/\text{day} \)
  b) Storage coefficient \( S = 0.0005 \)
  c) Aquifer thickness = 200 feet
  d) Well delivers a discharge of \( Q = 500 \text{ gpm} \)

• Determine the drawdown at an observation well located 150 feet away after \( t=12 \text{ hours} \) using Theis method. Use consistent equations.

(Note: 1 cfs = 448 gpm).
APPENDIX D

Unit Hydrograph

Additional Examples
Example: Unit Hydrograph Derivation

Problem Statement: Determine 1/2hr Unit Hydrograph using the excess rainfall hyetograph and Direct Runoff Hydrograph shown in the Table below. (Adapted from Chow et al (1988))

<table>
<thead>
<tr>
<th>Time ($\frac{1}{2}$ h)</th>
<th>Excess rainf. (in)</th>
<th>Direct runoff (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.06</td>
<td>428</td>
</tr>
<tr>
<td>2</td>
<td>1.93</td>
<td>1923</td>
</tr>
<tr>
<td>3</td>
<td>1.81</td>
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<tr>
<td>4</td>
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<td>10</td>
<td></td>
<td>830</td>
</tr>
<tr>
<td>11</td>
<td></td>
<td>313</td>
</tr>
</tbody>
</table>
Example: Unit Hydrograph Derivation (Cont.)

Solution by Deconvolution where \( N = M + J - 1 \) or \( J = N - M + 1 = 11 - 3 + 1 = 9 \) ordinates (Adapted from Chow et al (1988))

\[
U_1 = \frac{Q_1}{P_1} = \frac{428}{1.06} = 404 \text{ cfs/in}
\]

\[
U_2 = \frac{Q_2 - P_2 U_1}{P_1} = \frac{1923 - 1.93 \times 404}{1.06} = 1079 \text{ cfs/in}
\]

\[
U_3 = \frac{Q_3 - P_3 U_1 - P_2 U_2}{P_1} = \frac{5291 - 1.81 \times 404 - 1.93 \times 1079}{1.06} = 2343 \text{ cfs/in}
\]

and similarly for the remaining ordinates

\[
U_4 = \frac{9131 - 1.81 \times 1079 - 1.93 \times 2343}{1.06} = 2506 \text{ cfs/in}
\]

\[
U_5 = \frac{10625 - 1.81 \times 2343 - 1.93 \times 2506}{1.06} = 1460 \text{ cfs/in}
\]

\[
U_6 = \frac{7834 - 1.81 \times 2506 - 1.93 \times 1460}{1.06} = 453 \text{ cfs/in}
\]

\[
U_7 = \frac{3921 - 1.81 \times 1460 - 1.93 \times 453}{1.06} = 381 \text{ cfs/in}
\]

\[
U_8 = \frac{1846 - 1.81 \times 453 - 1.93 \times 381}{1.06} = 274 \text{ cfs/in}
\]

\[
U_9 = \frac{1402 - 1.81 \times 381 - 1.93 \times 274}{1.06} = 173 \text{ cfs/in}
\]

\( \frac{1}{2} \text{ hr UHG} \)

<table>
<thead>
<tr>
<th>( n )</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>( U_n ) (cfs/in)</td>
<td>404</td>
<td>1079</td>
<td>2343</td>
<td>2506</td>
<td>1460</td>
<td>453</td>
<td>381</td>
<td>274</td>
<td>173</td>
</tr>
</tbody>
</table>
Problem Statement:

The six-hour unit hydrograph of a watershed having a drainage area equal to 393 km² is as follows:

<table>
<thead>
<tr>
<th>Time (h)</th>
<th>0</th>
<th>6</th>
<th>12</th>
<th>18</th>
<th>24</th>
<th>30</th>
<th>36</th>
<th>42</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit hydrograph (m³/s·cm)</td>
<td>0</td>
<td>1.8</td>
<td>30.9</td>
<td>15.6</td>
<td>41.8</td>
<td>14.6</td>
<td>5.5</td>
<td>1.8</td>
</tr>
</tbody>
</table>

For a storm over the watershed having excess rainfall of 5 cm for the first six hours and 15 cm for the second six hours, compute the streamflow hydrograph, assuming constant baseflow of 100 m³/s.
Problem: Derivation of Direct Runoff and Streamflow Hydrographs Using a Given Unit Hydrograph (cont.)

Solution:

**Q3**

<table>
<thead>
<tr>
<th>Time (hr)</th>
<th>Rainfall</th>
<th>Excess (cm)</th>
<th>Runoff (cfs)</th>
<th>Streamflow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td>0</td>
<td>0</td>
<td>100.0</td>
</tr>
<tr>
<td>5</td>
<td>1.8</td>
<td>5</td>
<td>9.0</td>
<td>109.0</td>
</tr>
<tr>
<td>12</td>
<td>30.9</td>
<td>15</td>
<td>181.6</td>
<td>281.6</td>
</tr>
<tr>
<td>18</td>
<td>85.6</td>
<td>0</td>
<td>891.5</td>
<td>991.5</td>
</tr>
<tr>
<td>24</td>
<td>41.6</td>
<td>0</td>
<td>1493.0</td>
<td>1593.0</td>
</tr>
<tr>
<td>30</td>
<td>14.6</td>
<td>0</td>
<td>700.0</td>
<td>800.0</td>
</tr>
<tr>
<td>36</td>
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<td>0</td>
<td>246.5</td>
<td>346.5</td>
</tr>
<tr>
<td>42</td>
<td>1.8</td>
<td>0</td>
<td>81.5</td>
<td>181.5</td>
</tr>
<tr>
<td>48</td>
<td>0</td>
<td>0</td>
<td>27.0</td>
<td>127.0</td>
</tr>
<tr>
<td>54</td>
<td>0</td>
<td>0</td>
<td>0.0</td>
<td>100.0</td>
</tr>
</tbody>
</table>

\[
Q_1 = P_1U_1 = (5)(1.8) = 9
\]

\[
Q_2 = P_2U_1 + P_1U_2 = (15)(1.8) + (5)(30.9) = 181.5
\]

\[
Q_3 = P_3U_1 + P_2U_2 + P_1U_3 = 0 + (15)(30.9) + (5)(85.6) = 891.5
\]

\[
Q_4 = P_2U_3 + P_1U_4 = (15)(85.6) + (5)(41.8) = 1493
\]

**2nd Method**

<table>
<thead>
<tr>
<th>Time (hr)</th>
<th>Rainfall</th>
<th>Excess (cm)</th>
<th>U1</th>
<th>U2</th>
<th>U3</th>
<th>U4</th>
<th>U5</th>
<th>U6</th>
<th>U7</th>
<th>Direct</th>
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<td>0</td>
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</tr>
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<td></td>
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</tr>
</tbody>
</table>

Streamflow vs. Time

[Graph showing streamflow over time]
Example: Derivation of Direct Runoff and Streamflow Hydrographs Using a Given Unit Hydrograph

Problem Statement:

• The six-hour unit hydrograph of a watershed having a drainage area equal to 393 km² is as follows:

<table>
<thead>
<tr>
<th>Time (hr):</th>
<th>0</th>
<th>6</th>
<th>12</th>
<th>18</th>
<th>24</th>
<th>30</th>
<th>36</th>
<th>42</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit hydrograph (m³/s/cm):</td>
<td>0</td>
<td>1.8</td>
<td>30.9</td>
<td>15.6</td>
<td>41.8</td>
<td>14.6</td>
<td>5.5</td>
<td>1.8</td>
</tr>
</tbody>
</table>

• For a storm over the watershed having the following excess rainfall depths, compute the streamflow hydrograph assuming a constant base flow of 100 m³/s.

<table>
<thead>
<tr>
<th>Time (hr):</th>
<th>0</th>
<th>6</th>
<th>12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall Excess (cm):</td>
<td>0</td>
<td>5</td>
<td>15</td>
</tr>
</tbody>
</table>
Example: Derivation of Direct Runoff and Streamflow Hydrographs Using a Given Unit Hydrograph (cont.)

Solution:

<table>
<thead>
<tr>
<th>Time (hr)</th>
<th>UH (m³/s·cm)</th>
<th>Excess (cm)</th>
<th>Runoff (cfs)</th>
<th>Streamflow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.0</td>
<td>100.0</td>
</tr>
<tr>
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<td>1.8</td>
<td>5</td>
<td>9.0</td>
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<tr>
<td>12</td>
<td>30.9</td>
<td>15</td>
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<td>600.0</td>
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<td>27.0</td>
<td>127.0</td>
</tr>
<tr>
<td>54</td>
<td>0</td>
<td>0</td>
<td>0.0</td>
<td>100.0</td>
</tr>
</tbody>
</table>

Q₁ = P₁U₁ = (5)(1.8) = 9
Q₂ = P₂U₁ + P₁U₂ = (15)(1.8) + (5)(30.9) = 181.5
Q₃ = P₃U₁ + P₂U₂ + P₁U₃ = 0 + (15)(30.9) + (5)(85.6) = 891.5
Q₄ = P₄U₃ + P₁U₄ = (15)(85.6) + (5)(41.8) = 1493

2nd Method

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>Rainfall</th>
<th>Excess (cm)</th>
<th>U₁</th>
<th>U₂</th>
<th>U₃</th>
<th>U₄</th>
<th>U₅</th>
<th>U₆</th>
<th>U₇</th>
<th>Direct</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td></td>
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<td></td>
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<td>5</td>
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<td>181.5</td>
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Example: Unit Hydrograph Derivation

Problem Statement: Determine 1/2hr Unit Hydrograph using the excess rainfall hyetograph and Direct Runoff Hydrograph shown in the Table below. (Adapted from Chow et al (1988)

<table>
<thead>
<tr>
<th>Time (${\frac{1}{2}}$ h)</th>
<th>Excess rainf. (in)</th>
<th>Direct runoff (cfs)</th>
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<td>1</td>
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</tr>
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</tr>
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<tr>
<td>11</td>
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</tbody>
</table>
Example: Unit Hydrograph Derivation (Cont.)

Solution by Deconvolution where \( N = M + J - 1 \) or \( J = N - M + 1 = 11 - 3 + 1 = 9 \) ordinates

(Adapted from Chow et al (1988)

\[
U_1 = \frac{Q_1}{P_1} = \frac{428}{1.06} = 404 \text{ cfs/in}
\]

\[
U_2 = \frac{Q_2 - P_2U_1}{P_1} = \frac{1923 - 1.93 \times 404}{1.06} = 1079 \text{ cfs/in}
\]

\[
U_3 = \frac{Q_3 - P_3U_1 - P_2U_2}{P_1} = \frac{5291 - 1.81 \times 404 - 1.93 \times 1079}{1.06} = 2343 \text{ cfs/in}
\]

and similarly for the remaining ordinates

\[
U_4 = \frac{9131 - 1.81 \times 1079 - 1.93 \times 2343}{1.06} = 2506 \text{ cfs/in}
\]

\[
U_5 = \frac{10625 - 1.81 \times 2343 - 1.93 \times 2506}{1.06} = 1460 \text{ cfs/in}
\]

\[
U_6 = \frac{7834 - 1.81 \times 2506 - 1.93 \times 1460}{1.06} = 453 \text{ cfs/in}
\]

\[
U_7 = \frac{3921 - 1.81 \times 1460 - 1.93 \times 453}{1.06} = 381 \text{ cfs/in}
\]

\[
U_8 = \frac{1846 - 1.81 \times 453 - 1.93 \times 381}{1.06} = 274 \text{ cfs/in}
\]

\[
U_9 = \frac{1402 - 1.81 \times 381 - 1.93 \times 274}{1.06} = 173 \text{ cfs/in}
\]

\[
\begin{array}{|c|c|c|c|c|c|c|c|c|c|}
\hline
n & 1 & 2 & 3 & 4 & 5 & 6 & 7 & 8 & 9 \\
U_n \text{ (cfs/in)} & 404 & 1079 & 2343 & 2506 & 1460 & 453 & 381 & 274 & 173 \\
\hline
\end{array}
\]