Surface Water

Anyone familiar with their landscape will recognize water bodies that flow rapidly, such as rivers and streams, and those that flow imperceptibly, such as lakes, ponds, and wetlands. Moving water is called lentic, while calm water is called lotic. This chapter explores these components of the water cycle by explaining their sources and behavior.

Hydrographs

Both systems rise and fall as a result of hydrologic inputs and outputs, and this time-varying behavior is described using the hydrograph. Between storms, water levels normally decline slowly over time, rising in response to precipitation.

A stream hydrograph relates the discharge or stage of water as a function of time (Figure 1). The stream stage is the elevation of water in the channel, which normally increases as the discharge increases. The relationship between the stream stage and discharge is called the rating curve. A staff gage is a scale placed in the stream to measure the stream stage. The stream discharge is estimated by measuring the stream stage and then consulting the rating curve.

![Figure 1. Components of a hydrograph.](image)

Daily precipitation data should be plotted, along with daily evaporation. The difference between precipitation and evaporation can be compared to observed water levels. In systems where atmospheric exchanges dominate the hydrology, water levels rise during precipitation events, and fall at a rate controlled by the evaporation rate along with drainage from the watershed.

Between storms, streams normally decline slowly over time, rising in response to precipitation. The rising limb of the hydrograph corresponds to the period of time from when the stream stops declining until it reaches its peak. The peak discharge, or peak stage, corresponds to the time when the river reaches its highest level. The falling limb of the hydrograph corresponds to the period following the peak and lasts until the next storm.

The time to peak is the length of time between the peak precipitation and peak stage. Times to peak are short in urban areas with large impervious surfaces and channels that have been modified to increase stream velocities. Times to peak are longer in forested areas with few impervious surfaces and channels with many obstructions that slow the passage of water.

Hydrologists divide streamflow into three types of flow; stormflow, interflow, and baseflow. Stormflow refers to streamflow that occurs quickly in response to precipitation events. Interflow is a slower process that may take hours or days, while baseflow usually takes days to years to respond to rainfall. If a stream was flowing before the rainfall (a typical situation), stormflow is the flow that occurs in addition to the baseflow that would have occurred if it had not rained. There are many ways to separate streamflow into stormflow, interflow and baseflow.

The rising limb of the hydrograph corresponds to the period of time from when water levels begin to rise following a precipitation event. The peak stage, corresponds to the time when water levels reach their highest level. The falling limb of the hydrograph corresponds to the period following the peak and lasts until the next storm.

The time to peak is the length of time between the peak precipitation and peak stage. Times to peak are short in urban areas with large impervious surfaces and channels that have been modified to increase stream velocities. Times to peak are longer in forested areas with few impervious surfaces and channels with many obstructions that slow the passage of water.

Another term, the time of concentration, is the time required for flow to travel from the most distant point on the watershed, and is a function of the same factors that affect the time to peak.

Hydrologic measurements

Units

All quantitative studies require the definition of a measurement system -- but deciding which system to use is a universal problem! Here're two...

**English units.** The English system that we commonly use derives from an archaic system of measurement (Table 1). These conversions often depend on the type of good being measured, however, and may not be universally valid. Much like the conversion rate between the US dollar and other currencies, the conversion rate changed with time and good.

<table>
<thead>
<tr>
<th>Weights, mass, work</th>
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<tbody>
<tr>
<td>1 dry oz</td>
<td>437.5 grains</td>
</tr>
<tr>
<td>1 pound</td>
<td>16 dry oz = 7,000 grains</td>
</tr>
<tr>
<td>1 stone</td>
<td>14 lbs</td>
</tr>
<tr>
<td>1 slug (mass)</td>
<td>32.17 lbs</td>
</tr>
<tr>
<td>1 quarter (weight)</td>
<td>25 lbs</td>
</tr>
<tr>
<td>1 hundredweight</td>
<td>4 quarters</td>
</tr>
<tr>
<td>1 ton</td>
<td>20 hundredweight</td>
</tr>
</tbody>
</table>
1 horsepower = 550 ft-lbs/s

Length and area
1 yard = 3' = 36"
1 rod = 25 links = 16.5'
1 chain = 4 rods = 66 ft
1 furlong = 10 chains = 660 ft
1 mile = 8 furlongs = 5,280 ft
1 league = 3 miles = 15,840 ft
1 rood = 10 rods x 1 chain = 10,890 ft
1 acre = 1 furlong x 1 chain = 43,560 ft

Volume
1 tbsp = 3 tsp = 8 drams
1 fluid oz = 2 tbsp
1 gill = 4 fluid oz
1 cup = 2 gills
1 pint = 2 cups
1 quart = 2 pints
1 gallon = 4 qt
1 peck = 2 gal
1 kenning = 2 pecks
1 bushel = 2 kennings
1 strike = 2 bushels
1 pail = 2 strikes
1 chaldron = 4 pails
1 firkin = 9 gal
1 kilderkin = 2 firkins
1 barrel = 2 kilderkin
1 hogshead = 3 kilderkin
1 keg = 15.5 gal

<table>
<thead>
<tr>
<th>Volume</th>
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<tbody>
<tr>
<td>cubic meter (m³)</td>
<td>litre (L)</td>
</tr>
<tr>
<td>Time</td>
<td></td>
</tr>
<tr>
<td>second (s)</td>
<td></td>
</tr>
<tr>
<td>Energy</td>
<td></td>
</tr>
<tr>
<td>joule (J = kg·m²/s)</td>
<td></td>
</tr>
<tr>
<td>Power</td>
<td></td>
</tr>
<tr>
<td>watt (W = J/s)</td>
<td></td>
</tr>
<tr>
<td>Force</td>
<td></td>
</tr>
<tr>
<td>newton (N = J/m)</td>
<td></td>
</tr>
<tr>
<td>Pressure</td>
<td></td>
</tr>
<tr>
<td>pascal (Pa = J/m²)</td>
<td></td>
</tr>
</tbody>
</table>

Metric units. While English units are still used in two countries (the US and Liberia), all other countries use the metric system for official measurements (Table 2). Unfortunately, mixing English with other units can result in engineering failures, such as the destruction of the $125 million Mars Climate Orbiter spacecraft in 1999. In this case, two teams of scientists used different units, which were never reconciled.

Also, the adoption of the newer mks (meter-kilogram-second) system to replace the older cgs (centimeter-gram-second) system has not been widely accepted. The international metric system is used by most nations, and virtually all scientists. The international metric system of units was established to standardize units of measure (Table 2).

Table 2. Fundamental metric units

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>Mass</td>
<td>kilogram (kg)</td>
</tr>
<tr>
<td>Length</td>
<td>metre (m)</td>
</tr>
<tr>
<td>Area</td>
<td>hectare (ha)</td>
</tr>
</tbody>
</table>

Metric units are intended to be universal - independent of arbitrary human measurements. The meter was initially defined as a millionth of the distance from the north pole to the equator, measured along the longitude passing through Paris, France. The gram was defined as the mass of 1 mL (cm³) of water at its maximum density.

One of the more troublesome aspects of units are the plethora of different ways to measure pressure, shown in Table 3. Note that units of depth (cm, in) are dependent on the fluid density, which varies with temperature.

Table 3. Pressure units and conversions.

<table>
<thead>
<tr>
<th>1 bar</th>
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<tbody>
<tr>
<td>1,000 mbar</td>
<td></td>
</tr>
<tr>
<td>1,000 hPa</td>
<td></td>
</tr>
<tr>
<td>100 kPa</td>
<td></td>
</tr>
<tr>
<td>0.98692 atm</td>
<td></td>
</tr>
<tr>
<td>14.504 psi</td>
<td></td>
</tr>
<tr>
<td>0.980655 kg/m²</td>
<td></td>
</tr>
<tr>
<td>75.006 cm of Hg</td>
<td></td>
</tr>
<tr>
<td>29.613 in of Hg</td>
<td></td>
</tr>
<tr>
<td>401.19 in of H₂O</td>
<td></td>
</tr>
<tr>
<td>2.307 ft of H₂O</td>
<td></td>
</tr>
<tr>
<td>70.30 cm of H₂O</td>
<td></td>
</tr>
</tbody>
</table>

Suffixes and prefixes. These symbols are used to convert large units to small units, or vice versa. For example, 1 km = 1,000 m. Note that English units use suffixes, while metric units use prefixes (Table 4).

Table 4. Magnitude indicators for English (suffixes) and Metric (prefixes) units.

<table>
<thead>
<tr>
<th>English unit suffixes</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>ppt</td>
<td>10⁻¹²</td>
</tr>
<tr>
<td>ppb</td>
<td>10⁻⁹</td>
</tr>
<tr>
<td>ppm</td>
<td>10⁻⁶</td>
</tr>
<tr>
<td>%</td>
<td>10⁻³</td>
</tr>
<tr>
<td>‰</td>
<td>10⁻³</td>
</tr>
</tbody>
</table>

Metric unit prefixes
Monotoring water levels

Water levels are determined using a staff gage if the water level is above the ground surface. A staff gage is a vertical scale that serves to indicate the elevation of water, or stage, with respect to a reference elevation (Williams et al., 1996).

The staff gage is an inexpensive tool that should placed so that the base of the staff gage is always submerged in water. For ease of measurement, multiple staff gages can be placed at different depths, such that the nearest one is visible from the shoreline during wet weather.

The deeper staff gage is used once water levels fall below the nearer gage (Figure 2). In this way, one submerged staff gage is always visible from the shoreline as water levels rise and fall.

Significant digits. Significant digits are used to convey the accuracy implicit in a number; 31,345.78 ± 0.01 is a much more precise number than 31,000 ± 1,000. The number of non-zero digits in the first value is much greater than the second.

One would not report 31,345.78 ± 1,000 because all the non-zero digits after the first two are meaningless. For example, we can say π = 3.1, which has only two significant digits (as does 31,000), while π = 3.14 has three and π = 3.14159 has six, and π = 3.141592653589793 has 16 significant digits.

When reporting results, the number of digits to display must recognize the accuracy of the digits in the calculation. For example, 1/π = 0.318309886183791 has fifteen digits, while 1/3.1 = 0.322580645161290 falsely conveys that the accuracy is still to 15 digits, when in fact the accuracy is only to the first two digits, i.e., 1/3.1 = 0.32, because the value of π is only written to two digits.

This holds true regardless of the decimal place, i.e., 1/31 = 0.032, or 1/0.31 = 3.2, 1/0.031 = 32. For two numbers of varying accuracy, the reported number of digits is limited by the number with the fewest digits, i.e., π/3.1 = 1.0, rather than 1.013416985028966.

Yet, the digits should be preserved as much as possible when making calculations, because rounding during a calculation can reduce the accuracy of the final result. The final result should be simplified to the appropriate level of accuracy.

Measuring discharge

Surface water flows can be estimated using flow measurement control devices, such as weirs (which require a pool upstream and are not satisfactory when elevated sediment concentrations are present), flumes (which tend to flush sediments more effectively than weirs), and culverts (which are less accurate).

The relationship between stage and streamflow discharge is called the rating curve (Figure 3). Once a rating curve has been developed, the stream discharge is readily found by observing the stage and then consulting the rating curve.
Repeated measurements of discharge over a range of stages is required.

Control structures. Control structures avoid the vagaries of channel geometry by creating a uniform section. A common control structure is the weir (Figure 4), which has a stilling basin upstream of a constriction (normally called the weir blade), and a free-fall below the constriction. The stilling basin is used to eliminate the velocity head, yielding \( H = z \) in the stilling basin, where \( z \) is the measured water surface elevation above the lowermost point on the weir blade.

![Figure 4. Three types of weirs (rectangular, triangular, trapezoidal, flooded orifice) used as control structures for measuring stream discharge. Water level (stage) is measured in weir basin upstream of weir blade.](image)

Types of sharp-crested weirs include:

- **Flooded Orifice**: \( Q = C A H^{5/2} \)
- **Rectangular**: \( Q = C_a W H^{3/2} \)
- **Triangular**: \( Q = C_a \tan a H^{5/2} \)
- **Trapezoidal**: \( Q = C_a W H^{5/2} + C_a \tan a H^{5/2} \)

where \( H \) is the water surface elevation within the upstream weir pool relative to the invert elevation. Note that these equations neglect contraction effects along weir blade edges.

Two general types of weirs include broad-crested and sharp-crested. As the name implies, the broad-crested weir has a broad constriction in the direction of flow, while the sharp-crested weir has a knife-edge blade that forms the constriction.

A broad-crested weir consists of an outflow structure over which water flows for some distance before falling over the downstream edge. A sharp-crested weir is constructed so that the flowing water passes over a vertical, knife-edge, thus minimizing resistance with the weir blade.

Water flows out of the weir over the weir blade, which can take a variety of shapes, including triangular, rectangular, and trapezoidal. A submerged, circular orifice can also be used. The discharge is calculated using \( Q = v A = v(WH) \) where \( A = WH \) is the cross-sectional area perpendicular to flow above the weir blade, \( W \) is the width of the weir, and \( H \) is the depth of flow above the weir blade. So that the general weir formula is:

\[ Q = a H^5 \]

where \( a \) accounts for the cross-sectional area as well as contraction and energy losses, \( H \) is the water surface elevation in the stilling basin above the weir blade, and \( b = 0.5 \) for a flooded orifice, \( b = 1.5 \) for a rectangular weir, and \( b = 2.5 \) for a
triangular weir. This equation holds for the broad-crested weir as well, with \( b = 1.5 \).

Weirs may not provide accurate estimates in several situations. One source of error occurs when the weir blade becomes blocked by ice or floating debris, such as leaves and branches. Another source of error arises when the weir basin fills with sediments, resulting in an inaccurate estimate of the total head.

For all weirs, a staff gage or a water-level recorder is placed upstream of the constriction to measure the total head under subcritical conditions. Weirs tend to be more accurate than flumes, but suffer from sediment accumulation in the stilling basin, along with debris obstructing on weir crest.

For sediment-laden flows, a **flume** can be readily constructed with a uniform cross-sectional area, so that \( Q = W \ h \ v \) where \( W \) is the width of the flume, \( h \) is the depth of water in the flume, and \( v \) is the water velocity through the flume. In most cases, no unique relationship between depth and velocity can be established, being a function of the slope of the flume and the upstream and downstream conditions.

An improvement on the standard flume is to place a constriction in the flume (called the **throat**) that forces the flow to be subcritical (low velocity) upstream of the constriction, and supercritical (high velocity) downstream. Examples of this type include the **Parshall flume** and **H-Type flumes**.

The **Parshall flume** (**Figure 5**) is a high-precision flow measurement device that is commonly used in agriculture and water treatment plants. While able to pass small sediments and some woody debris, it is prone to clogging in the throat section.

![Figure 5. Parshall flume](image)

The H-type flume (**Figure 6**) was developed by the U.S. Department of Agriculture for measuring discharge in sediment-laden streams. Flumes require no upstream stilling basin, and allow sediment to pass unimpaired through the structure. Ice, leaves and other debris can still affect the reading, however.

![Figure 6. H-type flume used in sediment-laden streams.](image)

Culverts under roads can also be used as control structure. Four types of flow conditions can be found for most culverts; upstream intake either submerged or open, and downstream discharge conditions either submerged or open. When the upstream end is flooded, and the downstream end is open, then the orifice solution for the weir equation can be used.

When both upstream and downstream opening are submerged, then pipe flow conditions are present, and discharge can be found using the difference in head between the two ends, the culvert length and diameter, and the type of culvert (smooth, corrugated, etc.). When the upstream end is open and the downstream end is either open or flooded, then the flow can be found using indirect techniques such as the Manning's equation, below.

Regardless of flow conditions, it is better if the culvert has a uniform shape throughout its length and is not obstructed with debris. Elevation measurements can be made mechanically using a water level recorder, or visually using a staff gage.

Types of channel control structures include:

**Weirs:**
- stilling basin is located upstream of weir
- water level recorder is used to measure stage in stilling basin
- outlet structures include rectangular, triangular (v-notch), and Cipolletti (trapezoidal) shapes
- weir crests can be broad (flat lip) and sharp (knife-blade) crested
- flow is subcritical upstream of crest, supercritical downstream
- weirs collect sediments in the stilling basin, debris on weir blade

**Flumes:**
- no stilling basin, only a narrow throat
- regular approach section
- passes sediment easily
- large woody debris can be a problem

**Culverts:**
- Four combinations of flow equations, flooded vs. open upstream, flooded vs. open downstream
- Culvert should be a regular shape, round or rectangular, with no debris

**Indirect measurements.** For situations when control structures are not present, and instream measurements are not possible, then Manning's equation is commonly employed to indirectly measure water velocity:

\[
\text{Manning's Eqn: } v = \left(\frac{1}{n}\right) R^{2/3} S^{1/2}
\]

where \(n\) is the Manning's roughness coefficient, \(R\) is the hydraulic radius, and \(S\) is the gradient in total head. The roughness coefficient, \(n\), is normally found in tables, and is based on stream channel characteristics such as stream bed materials, amount of vegetation within the channel, variation in channel shape, and sinuosity.

The hydraulic radius, \(R = A/P\), is defined as the ratio of the stream cross-sectional area to the wetted perimeter, which is approximately equal to the water depth in a shallow, wide channel. The total head gradient is the drop in total head per unit distance of stream channel.

Once the average water velocity, \(v\), has been determined, the stream discharge, \(Q\), can be estimated using:

\[
Q = vA
\]

where \(A\) is the cross-sectional area of the channel.

**Estimating overbank flows.** Wetlands adjacent to riverine systems are often affected by overbank flows during stormflow periods. In these cases, the river spills out of its normal channel and overflows onto adjacent floodplains. The period of time that wetlands on the floodplains are affected by the duration of stormflow, and their amplitude. Instrumentation to monitor water levels in wetlands adjacent to riverine systems can be installed using techniques mentioned previously. Additionally, the U.S. Geological Survey provides estimates of overbank flooding frequencies for ungaged sites (Jennings, et al., 1994).

**Residence times**

The residence time is used to evaluate the time required for a hydrologic input to pass through the hydrologic system of interest. The residence time, \(\tau\), for a system with constant volume and constant flow rate is simply the ratio of the volume of water within the system, \(V\), to the flow rate, \(Q\), or:

\[
\tau = V / Q
\]

The estimated residence time is only appropriate for conditions of 1) piston-flow, such as a First-In-First-Out (FIFO) queue, 2) steady (i.e., constant) flow, 3) single locations of inflow and outflow, and 4) no atmospheric or groundwater exchanges (Himmelblau and Bischoff, 1968). The estimated residence time is not appropriate for conditions when water within the system mixes, when multiple inflows and/or exchanges occur at different points within the system, or when flow into the system changes over time.

If these conditions are not met, then the above equation only provides an estimate of the average residence time - actual residence times now varying over time and space. Functions for describing the distribution of residence times may be found for simple systems. For example, the exponential function can be used to determine the residence time distribution for a fully mixed system with constant inputs over time (Law and Kelton, 1991).

Different parts of a systems may exhibit different hydrologic residence times. Water in active, flow-through sections of a system may have shorter residence times than water in inactive, isolated parts. While each section may have identical hydropatterns, the flow is concentrated in one area, leaving other areas with stagnant conditions. The same equation can be applied regardless, in this case each section would be characterized using the volume of water present in the section, and the flow rate would be characterized using the flow into the section of interest.

Residence times for dynamic systems are more difficult to calculate than steady flow systems. In these cases, the residence time changes - with increasing residence times during periods when outflows exceed inflows, and decreasing when inflows exceed outflows. This is because adding so-called new water or removing old water from the system decreases the age.

While hydraulic residence times can be calculated using the above equation, tracer tests can also be conducted to confirm these calculations. Conservative tracers (i.e., non-reactive tracers that move passively with the water velocity) can be added at the inflow point, and then tracer concentrations can be monitored at the outflow location.

A **breakthrough curve** describes the resulting tracer concentration over time. The time required for the median concentration - when the outflow concentration equals half of the input concentration - provides an estimate of the average residence time of the system. The residence time is the time required for water to travel through a waterbody. A short residence time means that the water is flushed quickly, while a large residence time means that the water is isolated for a longer time.

**Nutrient cycling** is dependent, in large part, on the residence time because the time scales associated with food webs. Sufficient time for the various trophic levels is needed to develop and recycle the nutrients. If the water flows through a system faster than the nutrients can be cycled, then the reuse and accumulation of nutrients is not possible.

The average residence time, \(\bar{\tau}\), is calculated using the ratio of the water volume, \(V\), divided by the mean flow rate through the waterbody, \(Q\):

\[
\bar{\tau} = V / Q
\]
This means that a large system with a small outflow, such as Okefenokee Swamp in southeast Georgia, has a large residence time. Any pollutants added will take many years to be flushed out. On the other hand, a small lake with a large outflow, such as Lake Allatoona on the Etowah River in Georgia, has a shorter residence time, so that wastes are flushed out more readily.

The average residence time can be misleading, however. One problem is that the inflows and outflow may vary with time. For example, if there is no flow for part of the year, and then there is a large flood, then the residence time will be long during the drought, and become much shorter during the flood.

Another problem is that the average residence time assumes that all the water in the waterbody is completely mixed. If, however, the inflow bypasses the bulk of the water, such as by flowing around a pool in a channel, then the appropriate volume is much smaller.

We describe the different mixing scenarios using queueing theory:

**Mixed.** The water is completely mixed before it is discharged. This ensures that the outflow is randomly selected from the age distribution within the waterbody.

**FIFO: First-in First-out.** This is also called **piston flow,** because the first water that flows into the waterbody is the first water that flows out. It would be like waiting in line at a checkout counter. In effect, water must wait its turn behind other earlier water before it can discharge.

**LIFO: Last-in First-out.** This results when part of the water is isolated, and more active water bypasses the bulk of the water. It would be like getting on and off an airplane - the first ones on have to go to the back, and then wait until everyone else has gotten off. In a lake, water may skim across the surface, bypassing the deeper, more stagnant water at the bottom of the lake.

We can also talk about a residence time **distribution,** or the range of residence times for water in the waterbody. To determine the residence time distribution, one can inject a tracer, say into the primary inflow, and then observe the concentration over time in the discharge. Alternatively, one could mix the tracer uniformly throughout the waterbody, and then monitor the discharge concentration over time.

Each of these approaches assumes a **static condition,** i.e., the flow or amount of stratification does not change with time. Realistically, the residence time distribution is a **dynamic condition,** that changes with inflow and the amount of stratification within the waterbody.

**Hydropattern**

The temporal variability of water levels in streams results from dynamic changes in hydrologic inputs and outputs, and temporal changes associated with hydraulic controls within the stream channel. Temporal changes in water level are important determinants for many aquatic flora and fauna. The reproductive success of these species can be adversely affected when fluctuations are not correctly synchronized with their developmental stages.

The **hydropattern** is a distinctive feature of the hydrologic variability that describes the variation of water levels over time and space (Acosta and Perry, 2001; King et al., 2004). Hydropattern is a recent term that is used to expand the traditional concept of **hydropériod** (i.e., the frequency and duration of time at a given stage, Figure 7) by incorporating additional information about the aerial extent and timing of inundation. The aerial extent is important, especially for large, complex systems that contain a variety of watershed features.

![Figure 7](image)

**Figure 7.** Hydrograph for short period of time showing the water level variation. Note that the hydroperiod is marked for a few stages.

Several approaches can be used to characterize temporal changes in stage (i.e., water levels). The easiest approach is to plot stage as a function of time and shows the stage for a period of time that captures the range of possible hydrologic variability. Inter-annual, seasonal, event, and daily water level fluctuations may become apparent using such an approach.

A plot of flooding duration versus stage can be constructed using observed water levels (Figure 8). This plot provides a descriptive summary that indicates how long a typical flood occurs for each stage. Lower elevations have longer durations of flooding than higher elevations.

![Figure 8](image)

**Figure 8.** Left: Hydropériod plot showing duration of flooding versus stage. Right: Stage-frequency plot showing number of exceedances per year. Note that the longest duration of flooding occurs at the lower stages, and vice versa. Lower stages have higher frequency of being flooded than higher stages.

This approach is useful for characterizing water level variability by generating a stage-duration relationship that quantifies the duration in time that a specified water level is exceeded. In this
case the period of time that water levels exceed the specified stage (or range in stages) is described. This approach should also consider the seasonal nature of inundations by dividing the data into specific time frames (Mitsch and Gosselink, 2000).

While the stage-duration approach successfully captures the duration of time that the system is flooded, it fails to characterize the frequency with which this occurs. That is, the number of times that a water level exceeds a specified stage for a specified period of time is not quantified.

An alternative approach is to quantify the frequency in time that the stream is observed to exceed a range of specified stages. This approach yields a cumulative frequency table or plot that can be used to calculate exceedance probabilities. The mean, median, and extreme stages (e.g., 1, 10, 50, 90, and 99 percentile probability) can be estimated using the exceedance probability plot (McCuen, 1998).

A significant drawback using the exceedance probability approach is that the correlation structure between individual observations may or may not be captured. That is, a system whose water levels vary slowly over time can display the same frequency distribution of water levels as does a system that varies quickly.

In effect, the amplitudes of the fluctuations are described, but not the duration. An additional problem is that the frequency diagram, in aggregate, may poorly convey daily and seasonal behavior. Partitioning or stratifying data sets into seasonal or other periods may improve the characterization of water level conditions (Mitsch and Gosselink, 2000).

To overcome these limitations, a stage-duration-frequency, or SDF, curve can be constructed. The SDF plot is analogous to intensity-duration-frequency (IDF) curves used in precipitation analysis (McCuen, 1998). SDF curves indicates the frequency that a depth-duration relationship is observed (Figure 9).

The hydrodynamic behavior may be different from one stream to the next, as well as between stream segments. Characterizing the hydropattern for the entire watershed is a substantially more difficult exercise than for a small, uniform area.

**Stormwater**

The hydrologic response to precipitation is one of nature’s wonders. In forested watersheds the runoff is gentle - delayed by thick layers of litter and roots that hold the soil and water in place. In urban watersheds, the water rushes unchecked into pipes and concrete-lined channels that convey it rapidly to streams, carrying with it the litter and feces of countless people and pets.

Stormwater is the rapid response of a stream to a precipitation event. Unlike baseflow, which responds slowly to atmospheric inputs, stormwater responds immediately -- rising and falling quickly with the weather. Evidence of stormwater in an urban environment is easy to obtain -- puddles, flooded streets, and raging storm sewers. Evidence in agricultural settings is also readily seen -- sheet flow (i.e., overland flow), flooded fields, and rill erosion.

Overland flow in urban and agricultural environments results from low infiltration rates due to pavement and soil compaction. This flow type of overland flow is called *surface runoff* or Hortonian flow.

Finding stormwater sources in forested areas is more difficult, however, because of the higher permeability of forest soils. Water readily infiltrates into worm and animal burrows, and into decayed roots channels. Even though the water is underground, it can still move quickly through macropores to nearby streams. This flow, called *subsurface runoff*.

Yet, overland flow may occur in forested environments when sufficient rainfall causes saturation of the underlying soil. In this case, it is not a reduction in soil permeability that inhibits infiltration, but rather the reduction of the hydraulic gradient into the soil. Subsurface saturation essentially raises the water table to the ground surface, causing near-hydrostatic conditions in the subsurface.

This chapter deals with the effects of watershed alteration on stormflow. This is a concern due to the adverse peak flows, as well as degraded water quality, both of which are harmful to aquatic ecosystems. Let us first examine the physical flow of water, putting aside the chemical and biological aspects aside until a later chapter. We can look at what causes stormwater, and then examine how land uses alter stormwater behavior. We shall see how the alteration of landscapes from forest to agriculture to urban uses increases the magnitude and decreases the timing of flood peaks.

**Stormwater generation**

The total volume of stormwater is often a source of concern, especially for reservoir and detention basins that are designed
to contain a specific volume of water. In this case, the runoff rate is not of concern — only the total inflow volume:

\[ V = \sum Q(t) \Delta t \]

where \( V \) is the total runoff volume, \( Q(t) \) is the runoff at time \( t \), and \( \Delta t \) is the duration over which the runoff is measured. For example, if \( Q = 100 \text{ L/day} \) and \( \Delta t = 2 \text{ days} \), then \( V = 200 \text{ L} \).

The runoff volume can be translated to an equivalent watershed depth, \( D_w = \frac{V}{A_w} \), by dividing the total volume by the watershed area, \( A_w \). For example, if the total runoff volume is 10,000 L = 10 m\(^3\) and the watershed area is 1 ha = 10,000 m\(^2\), then the runoff depth is \( D_w = 1 \text{ mm} \).

Alternatively, the runoff volume can be translated to a total storage depth in a reservoir, \( D_r = \frac{V}{A_r} \), by dividing by the reservoir area. For a reservoir with an area that changes with lake water level, then an Area-Stage-Volume curve is used to convert between them.

Two fundamentally different models of stormwater generation have been proposed. The first posits that runoff occurs when the rainfall intensity exceeds the soil infiltration rate (Figure 10). In this model, high intensity rains are more likely to cause stormflow than low intensity storms.

Figure 10. Excess precipitation resulting from limited soil infiltration capacity.

A second approach posits that soil storage limits the volume of water that the soil can absorb. In this model, storms with more rainfall depth, as opposed to greater rainfall intensities, result in greater runoff. Areas that are saturated contribute to stormflow, while areas that are unsaturated absorb all the rainfall and do not contribute to stormflow (Figure 11).

Figure 11. Excess precipitation resulting from limited soil moisture storage.

Curve Number

The Curve Number method is a widely used approach for estimating the runoff from different landscapes. The assumed relationship between storm effective rainfall depth, \( P \) and runoff depth, \( Q \), is:

\[ \frac{Q}{P} = \frac{F}{S} = \Theta \]

where \( F = P - Q \) is the abstraction depth, \( S \) is the maximum soil storage depth, and \( \Theta = F/S \) is the relative saturation of the soil, which varies between \( 0 < \Theta < 1 \).

Replacing the abstraction depth and solving for the runoff depth results in:

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

To use the curve number method, we first find the effective precipitation, \( P = P_o - I_a \), where \( P_o \) is the observed precipitation and \( I_a = 0.2 \text{ S} \) is the initial abstraction of water required for runoff to begin. Combining both Equations yields:

\[ Q = P^2 / (P + S) = (P_o - I_a)^2 / (P_o - I_a + S) \]

We then use the curve number, \( CN \), to find the maximum storage capacity of the soil, \( S \), in units of inches:

\[ S = 1000 / CN - 10 \]

Curve Numbers are obtained using the type of land use, vegetation, and soil factors as variables. Example curve numbers are shown in Table 5, and a plot of runoff vs rainfall is shown in Figure 12.
Table 5. Curve Numbers for common land uses.

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Hydrologic Soil Group</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>Woods and forests</td>
<td></td>
</tr>
<tr>
<td>Good; no grazing, brush</td>
<td>30</td>
</tr>
<tr>
<td>Fair; grazed, not burned; some brush</td>
<td>36</td>
</tr>
<tr>
<td>Poor; over-grazed, burned</td>
<td>45</td>
</tr>
<tr>
<td>Open spaces (lawns, parks, golf courses, cemeteries)</td>
<td></td>
</tr>
<tr>
<td>Good; &gt;75% grass</td>
<td>39</td>
</tr>
<tr>
<td>Fair; 50-75% grass</td>
<td>49</td>
</tr>
<tr>
<td>Pasture and range</td>
<td></td>
</tr>
<tr>
<td>Brush; &gt;75% ground cover</td>
<td>30</td>
</tr>
<tr>
<td>Meadow; grass, no grazing, mowed for hay</td>
<td>30</td>
</tr>
<tr>
<td>Good; 50-75% ground cover, not heavily grazed</td>
<td>39</td>
</tr>
<tr>
<td>Poor; &lt;50% ground cover, over-grazed</td>
<td>68</td>
</tr>
<tr>
<td>Cultivated</td>
<td></td>
</tr>
<tr>
<td>With conservation treatment (terraces, contour plowing)</td>
<td>62</td>
</tr>
<tr>
<td>Without conservation treatment (no terraces, contour plowing)</td>
<td>72</td>
</tr>
<tr>
<td>Residential</td>
<td></td>
</tr>
<tr>
<td>1 acre lots, 20% impervious</td>
<td>51</td>
</tr>
<tr>
<td>½ acre lots, 25% impervious</td>
<td>54</td>
</tr>
<tr>
<td>1/4 acre lots, 38% impervious</td>
<td>61</td>
</tr>
<tr>
<td>1/8 acre lots, 65% impervious</td>
<td>77</td>
</tr>
<tr>
<td>Industrial, 72% impervious</td>
<td>81</td>
</tr>
<tr>
<td>Business, 85% impervious</td>
<td>89</td>
</tr>
<tr>
<td>Roads</td>
<td></td>
</tr>
<tr>
<td>Dirt</td>
<td>72</td>
</tr>
<tr>
<td>Gravel</td>
<td>76</td>
</tr>
<tr>
<td>Paved with curbs and gutters</td>
<td>98</td>
</tr>
<tr>
<td>Roofs, driveways, paved parking</td>
<td>98</td>
</tr>
</tbody>
</table>

Figure 12. Curve Number method for estimating stormwater runoff

While this table is commonly used for design purposes, prediction uncertainties are large. The precautionary principle should be used; estimates of the expected as well as upper and lower bounds should always be considered. From a practical standpoint, adding and subtracting 20 to the tabulated Curve Number will account for the likely range of hydrologic responses to precipitation.

The hydrologic soil groups used in the Curve Number table are:

- **Soil Group A**: High infiltration (low runoff); Sand, loamy sand, or sandy loam; Infiltration rate greater than 1 cm/hr when wet.
- **Soil Group B**: Moderate infiltration (moderate runoff); Silt loam or loam; Infiltration rate 0.5 to 1 cm/hr when wet.
- **Soil Group C**: Low infiltration (moderate to high runoff); Sandy clay loam; Infiltration rate 0.1 to 0.5 cm/hr when wet.
- **Soil Group D**: Very low infiltration (high runoff); Clay loam, silty clay loam, sandy clay, silty clay, or clay; Infiltration rate less than 0.1 cm/hr when wet.

Antecedent moisture conditions are also important, in that they increase the amount of water stored in the soil, thus increasing the water content and reducing the amount of storage available in the soil. Three types of moisture conditions are generally used:

- **Dry**: Soils are dry, but not to the wilting point - decrease the CN
- **Normal**: Typical, or average, conditions - use the standard CN
- **Wet**: Recent heavy rainfall with saturated soils - increase the CN

To illustrate the curve number concept, let us define the total runoff efficiency, \( R = \frac{Q}{P} \), as the ratio of the total depth of runoff, \( Q \), to the total precipitation depth, \( P \).

The usual limits of the total runoff efficiency, \( 0 < R < 1 \), reflect the fact that stormwater runoff does not exceed rainfall, and can not be negative. During a storm, the runoff efficiency is commonly observed to increase as soils saturate.
At any given time, we can find the precipitation rate using:
\[ p = \frac{dP}{dt} = \frac{\Delta P}{\Delta t} \]
which is just the change in precipitation depth per unit time. Likewise, the runoff rate is
\[ q = \frac{dQ}{dt} = \frac{\Delta Q}{\Delta t} \]

The runoff rate, \( q \), can be calculated using:
\[ q = p\left(0 \times A_1 + 1 \times A_2 \right) = p a_2 \]

where \( A_1 \) is the area where the rainfall rate is less than the infiltration rate, \( A_2 \) is the area of impervious surfaces, and \( a_2 = \frac{A_2}{A_1 + A_2} \) is the proportion of area that is impervious.

This equation implies that runoff is not generated in areas where the rainfall is less than the infiltration rate, because all of the rainfall is absorbed into the soil. It also implies that, once the soil is saturated, all of the rainfall is converted into runoff.

This is the essence of the contributing area concept, which posits that only those saturated parts of the landscape that are interconnected with streams actually contribute to stormflow.

We can pursue this concept by defining an abstraction, or infiltration, rate, \( f \), as:
\[ f = p - q = p(1 - a) = p a_1 \]

where \( a_1 \) is the proportion of pervious surfaces. This means that all infiltration occurs only in those areas which are not saturated.

While some residual infiltration may occur in saturated areas, many saturated areas can actually be exfiltrating, or discharging water from the subsurface to the surface, so that the net effect of infiltration plus exfiltration may balance each other.

One can calculate the runoff efficiency, \( r \), from this function as:
\[ r = \frac{q}{p} = \frac{dQ}{dP} = 2P/(P+S) - P/\left(P+S\right)^2 = 1 - (1 + P/S)^{-2} = 1 - (1 - \Theta)^2 \]

which is shown in **Figure 13**. Note that the runoff efficiency increases rapidly with water content, and then asymptotically approaches one as the watershed nears saturation.

Note that other functions can also be used to represent the relationship between runoff and soil moisture storage. One approach would be to assume that soils are spatially variable across the landscape, and then assign a unique distribution of moisture storages to the soil.

**Peak flows**

Peak discharge, \( Q_p \), is another widely used measure of stormwater impact because the maximum damage is commonly associated with the peak magnitude. Roads are washed away - and worse yet, lives are lost as cars and homes are carried away by the swirling floodwaters.

While runoff is generated at various points across the landscape, stormwater discharge is only measured at a single point downstream. Thus, the rate at which stormwater is generated across the landscape must be converted into a point discharge. We do this by incorporating the runoff travel time between the source and the measurement location. There are several methods for estimating peak discharges from small watersheds - i.e., from catchments less than approximately 40 ha.

**Rational method**

Empirical models can be used to predict peak flows, \( Q_p \), using watershed data. Probably the simplest method is the so-called Rational method:
\[ Q_p = C i A \]

where \( C \) is a land cover coefficient, \( i \) is the rainfall intensity for duration equal to time of concentration, \( t_c \), and \( A \) is the watershed area. Values for the runoff coefficient, \( C \), for different land covers are presented in **Table 6**.

**Table 6. Rational method runoff coefficient, \( C \), for different land covers.**

<table>
<thead>
<tr>
<th>Land Cover</th>
<th>Low</th>
<th>High</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lawns</td>
<td>0.05</td>
<td>0.35</td>
</tr>
<tr>
<td>Forest</td>
<td>0.05</td>
<td>0.25</td>
</tr>
<tr>
<td>Cultivated land</td>
<td>0.08</td>
<td>0.41</td>
</tr>
<tr>
<td>Meadow</td>
<td>0.10</td>
<td>0.50</td>
</tr>
<tr>
<td>Parks, cemeteries</td>
<td>0.10</td>
<td>0.25</td>
</tr>
<tr>
<td>Unimproved areas</td>
<td>0.10</td>
<td>0.30</td>
</tr>
<tr>
<td>Pasture</td>
<td>0.12</td>
<td>0.62</td>
</tr>
<tr>
<td>Residential areas</td>
<td>0.30</td>
<td>0.75</td>
</tr>
<tr>
<td>Business areas</td>
<td>0.50</td>
<td>0.95</td>
</tr>
<tr>
<td>Industrial areas</td>
<td>0.50</td>
<td>0.90</td>
</tr>
<tr>
<td>Streets -- bricks</td>
<td>0.70</td>
<td>0.85</td>
</tr>
</tbody>
</table>
The time of concentration, $t_p$, is the time of travel for stormwater runoff from the most distant point in the watershed to reach the outlet. If the watershed is $L = 600$ m in length, and the stormwater velocity is $v = 1$ m/s, then the time of concentration is $t_p = L/v = 600$ s or 10 min. The time of concentration is used for selecting the appropriate precipitation event because the largest runoff peaks are associated with storm events of this duration.

Note that the peak discharge is linear with the watershed area, so that we can write:

$$q_p = Q_p / A = C i$$

where $q_p$ is the runoff per area, which is a water yield - where a yield is an amount of some material (water, pollution, etc.) per unit area.

**Watershed characteristics**

Note that the rational method does not directly account for variations in topography or hydrographic features, such as lakes and ponds. More elaborate empirical models can be used, such as the Benson model:

$$Q_p = a A^c S^e S_i^g I_p^h L^i$$

where $A$ is the watershed area, $S$ is the main channel slope, $S_i$ is the area of lakes and ponds, $I_p$ is the maximum 24-hour, 10-year precipitation depth, and $L$ is the main channel length. Another model, proposed by Scott, is:

$$Q_p = a A^c E_m^c S_i^d P_s^e T^g$$

where $E_m$ is the mean altitude, $P_s$ is the average May to September precipitation depth, $I$ is the maximum 24-hour, 2-year precipitation depth, and $T$ is the mean January temperature. And finally, the Borland model is:

$$Q_p = a A^j E_m^c S_h^d S_i^d P_s^e P^h L_i^j L_o^k$$

where $S_h$ is the watershed shape, $P_s$ is the average October to April precipitation depth, $L_i$ is the latitude, and $L_o$ is the longitude.

Each of these models is formed by using peak stormflow at gages where watershed information is available. Normally, these models are regional - in that they can only be applied in the area where they were developed.

To form your own model, logarithmic transforms are used to linearize the equations:

$$y = x_0 + b x_1 + c x_2 + d x_3 + ...$$

where $y = \log Q_p$, $x_0 = \log a$, $x_1 = \log A$, etc.

Note that any number of factors can be added, thus improving the fitting or calibration error. Multicollinearity is a major problem with this approach, however, causing large prediction errors. One must always check the prediction uncertainties to make sure that each added parameter further decreases the prediction error.

In Georgia, the US Geological Survey has developed regional flood prediction equations for both urban and rural watersheds.

**Channel geometry**

Another simple model of peak runoff uses the channel width, $w$:

$$Q_p = a w^b$$

This approach assumes that the channel width is in equilibrium with peak discharges, so that larger peak flows are reflected in larger channel widths.

The channel geometry approach is not appropriate when the characteristics of the contributing area change over time. This is because the channel may not immediately respond to upstream alterations. As storm peaks increase - due to development, for example - then the channel will slowly widen until a new equilibrium is achieved. Likewise, if a degraded watershed is restored using riparian buffers and stormwater retention basins, then the channel will slowly establish a new equilibrium shape.

**Unit hydrographs**

Unit hydrographs are used when the shape of a stormwater hydrograph is desired. It is used to predict the stormflow hydrograph for conditions where one unit of effective precipitation (net runoff) falls on a watershed during one time period. The duration, time to peak, and peak discharge are all represented using a unit hydrograph.

A common shape to use is the triangular unit hydrograph - where the duration of the hydrograph, $t_c$, is the time of concentration within the watershed, and is the base of the triangle (Figure 14). The peak discharge, $Q_p$, is the height of the triangle, and the maximum height occurs at the time to peak, $t_p$.

The area of the triangle, $Q = t_c Q_p / 2$, represents the total volume of stormwater runoff. We normally set the total volume equal to one unit of runoff, hence the name unit hydrograph. Note that reducing the time of concentration requires a higher peak in order to maintain a unit runoff.
Other shapes besides a triangle can certainly be used - the only constraint is that the area under the curve must equal zero. The basic concept that a unit of runoff has a specific shape that is determined by the watershed.

**Linear systems**

A linear system is composed of inputs, outputs, state variables, and parameters. In this case, the input is the observed precipitation, the output is the predicted hydrograph, the state variables are watershed characteristics such as antecedent moisture conditions, and the parameters are the properties of the unit hydrograph.

Once a unit hydrograph has been constructed, we then use linear theory to manipulate the unit hydrograph for various precipitation events:

**Linear**: Linear theory assumes that responses are *linear* - a doubling of the net precipitation causes a doubling of the predicted stormwater discharge.

**Time invariant**: We also assume that the system is *time-invariant* - the response to precipitation in one hour is the same as the response in any hour.

**Independent**: We must also assume that the responses are *independent* of each other - the runoff in one hour does not affect the runoff in another hour.

For example, doubling an input doubles the response:

\[ y = f(x), \text{ then } cy = c f(x) = f(cx) \]

Note that \( y = mx + b \) is not a linear system because \( 2(mx+b) \) does not equal \( m(2x) + b \).

If the system is linear, then an input that is different than unity means that the output can be simply scaled by the input. Thus, if we had two units of net precipitation, then we would double the volume of the unit hydrograph by doubling the discharge heights while holding the time base constant.

**Convolution**

Convolution is used to scale and lag the unit hydrograph to reflect changing rainfall intensities over time. For example, two units of rainfall at the same time cause twice the runoff, but no change in lag, while two units of rainfall at different times, are the sum of individual rainfall events, where the two events are lagged by the delay between the two events.

Mathematically, the convolution operation is defined using the *star operator*, \(*\):

\[
y = h * x = \int_{-\infty}^{t} h(i) x(t-i) \, di = \sum_{i=-\infty}^{t} h(i) x(t-i)
\]

where \( y \) is the system response, \( h \) is the unit response function, \(*\) is the convolution operator, \( x \) is the input, and \( i \) is the time lag.

The three steps involved are:

1. Multiply unit response function, \( h(i) \), by input, \( x(t) \), for time step \( t \)
2. Shift to next hour and repeat Step 1, starting response at beginning of input interval
3. When all inputs have been multiplied by the unit hydrograph, add all responses.

An example of this method is presented in Table 7.

### Table 7. Unit hydrograph example.

<table>
<thead>
<tr>
<th>Time, t</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>Sum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input, x</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Lag</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>Sum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Response</td>
<td>0.0</td>
<td>0.6</td>
<td>0.3</td>
<td>0.1</td>
<td>0.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

### Hydrologic Response

<table>
<thead>
<tr>
<th>Hourly Response</th>
<th>Hourly Rainfall</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>3</td>
<td>0.6</td>
<td>0.0</td>
</tr>
<tr>
<td>4</td>
<td>0.3</td>
<td>1.2</td>
</tr>
<tr>
<td>5</td>
<td>0.1</td>
<td>0.6</td>
</tr>
<tr>
<td>6</td>
<td>0.0</td>
<td>0.2</td>
</tr>
<tr>
<td>7</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>8</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>1.0</strong></td>
<td><strong>2.0</strong></td>
</tr>
</tbody>
</table>

Estimating the unit hydrograph

To find the unit hydrograph, we need to have two time series, one of rainfall and the second of streamflow. Ideally, we would
use data that are similar to the ideal prediction conditions, also call the design storm.

This means that we should use data from flood periods if we wish to predict a unit hydrograph that is appropriate for flood conditions, and data from drought conditions if we wish to create a unit hydrograph appropriate for drought conditions. In other words, given that the unit hydrograph is probably dependent upon a state variable, it is best to use data sets for which the state variable is most similar to the prediction requirements.

To estimate the coefficients of the unit hydrograph, we construct a regression deconvolution equation of the form:

\[ y(t) = h(0) x(t-0) + h(1) x(t-1) + h(2) x(t-2) + \ldots + h(n) x(t-n) \]

where \( n \) is the temporal memory of the system, equal to the time of concentration.

Various methods can be used to estimate the unit hydrograph without streamflow data, including Snyder's, SCS Methods, Gray's, Expey's, Clark's, Nash's, and the Colorado method.

**Lentic systems**

Smaller than an ocean or sea, larger than a pond, deeper than a wetland, lakes are an important part of the hydrologic environment.

The shoreline margin of a lake is called the littoral zone. In this zone, the water is shallow enough that rooted aquatic plants can reach the water surface. Dense vegetation can be found in the littoral zone, but only when water levels levels remain fairly constant, and wave action does not erode them.

Further from shore, the profundal zone is an open-water zone where the water depth is too great for rooted aquatic plants to reach the surface. In this region, vegetation may form mats that float on the surface, or benthic rooted aquatics may be found on the lake bottom. Benthic aquatics, however, are limited by the amount of light that can reach the bottom, and by a substrate to which they can attach. The benthic zone may be composed of rocky, sandy, muddy, or organic mucks, depending upon the wave energy in the lake and the amount of organic matter that settles to the bottom.

Within the water column, pelagic, or suspended, plants and animal communities dominate. These organisms have densities close to water, and either float with the water currents, or have their own means of propulsion that allows them to maneuver.

**Stratification**

Lentic systems stratify vertically due to their lack of mixing. As a result, distinct layers form within the water column with different physical, chemical, and biological properties, depending on the amount of sunlight, nutrients, food supply and temperature.

Stratification often results from variations in fluid densities, with the most dense water at the bottom, and the least dense on the surface. The surface layer is called the epilimnion, while the deeper (benthic) layer is called the hypolimnion.

One mechanism for developing density differences occurs when surface water is warmer than benthic water. During the summer, the sun’s energy warms the surface and the underlying (colder) water gets trapped because it is denser. Also, the surface layer tends to be better mixed by wind than the underlying benthic layer.

Soil materials underneath the lake bottom moderate water temperatures, by conducting heat upward when the lake is colder, and conducting heat downward when the lake is warmer. This influence is greater in the benthos. Groundwater inflows also stabilize lake water temperatures because of the large thermal mass of the subsurface.

Another mechanism occurs during the winter when the surface cools to freezing. Because freshwater has a maximum density near 4°C, water colder than this temperature is lighter, and tends to float on the surface. Thus, rather than the epilimnion being warmer than the hypolimnion, during winter the epilimnion can be colder than the hypolimnion.

Yet another mechanism for stratification occurs when dissolved salts are concentrated. The salinity of the water increases the density of the water, causing it to sink. Thus, water high in salts tends to sink to the bottom, while fresh water tends to float on the surface.

An interesting application is predicting at what depth an inflow to a lake will migrate. In general, the inflow will settle to the depth at which its density is the same as the lake’s.

In the summer, the bottom of many lakes is quite cold, say 5°C, while the surface is much warmer, say 30°C. If the inflow temperature is 15°C, then the inflow water will settle below the surface of the lake, but above the bottom of the lake, at a depth where the lake temperature is approximately equal to the inflow temperature.

Later in the fall, once the bottom of the lake has warmed, say to 20°C, then the same inflow will settle to the bottom. In the spring, once the whole lake has cooled, say to 5°C, then the inflow will be found floating on the surface of the lake.

**Lake Lanier** is a reservoir impounded by Buford Dam on the Chattahoochee River in north-central Georgia.

- **Drainage Area** = 270,000 ha
- **Annual Discharge** = 58,800 L/s
- **Mean Residence Time** = 466 days

The reservoir is divided into four management zones (Table 8). The purpose of these zones is to allocate water for different purposes.

**Inactive Pool**: The lowermost zone that is not intended for storage, and is reserved for sediment accumulation.

**Conservation Pool**: The primary management zone that is used for storing water for multiple purposes, including:

- municipal water supply and wastewater dilution in the Atlanta Metropolitan area downstream
- downstream navigation
- hydroelectric power production

**Flood Control Pool**: used to manage stormflow for preventing damage along the river corridor below Atlanta.

**Maximum Pool**: Used only for emergency use during catastrophic flow conditions.

---

**Table 8. Lake Lanier management zones.**

<table>
<thead>
<tr>
<th>Pool</th>
<th>Elevation (ft)</th>
<th>Area (acres)</th>
<th>Volume ($10^3$ AF)</th>
<th>Cumulative ($10^3$ AF)</th>
</tr>
</thead>
<tbody>
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**Homework**

**A. Units**

1. A problem with the metric prefix arises when there is an exponent on the unit. Which of the following two choices are correct?
   - $1 \text{ km}^3 = 1 \text{k(m}^3) = 1,000 \text{ m}^3$
   - $1 \text{ km}^3 = 1 \text{(km)}^3 = 1,000,000,000 \text{ m}^3$

**B. Measuring streamflow**

1. What might be the advantages of a triangular vs. a rectangular weir?
2. What might be the advantages of a sharp- vs. a broad-crested weir?
3. Find the discharge from a culvert with a diameter of 1 m, a length of 10 m, and a drop of 1 m. Assume that the culvert is half full throughout its entire length and has a Manning coefficient of $n = 0.02$.
4. What does a rating curve of $Q = a h^{1.49}$ tell you about the channel shape?
5. How would changing Manning’s coefficient affects onsite vs offsite (downstream) flooding?

**C. Residence times**

1. Consider a well mixed lake containing water that is 30 days old on March 1. Find the residence time of lake outflows on March 30 assuming:
   - The lake has no inflows
   - No inflows between March 1 and March 29, but a large storm replaces half of the volume of the lake early in the morning of March 30
   - A large storm replaces half the lake volume on March 1, but there are no additional inflows for the rest of the month
   - Lake inflows account for one percent of the lake volume every day of the month

**D. Curve Number method**

1. Write and define the terms in the Curve Number Equation.
2. Explain how reducing the soil moisture storage capacity from 10 to 1 cm affects the volume of stormwater generated.
3. Calculate the Curve Number for soil moisture storages of 0, 1, 3, 5, 10, 30 and 50 cm.
4. Calculate the maximum soil moisture storage for Curve Numbers of 70, 80, 90, and 95.
5. Find the equation for the change in stormwater runoff per unit change in precipitation, dQ/dP
6. Find the equation for the change in stormwater runoff per unit change in maximum storage, dQ/dS
7. Plot dQ/dS for a range of S (e.g., S = 1, 5, 10, 50 cm)
8. Plot dQ/dS for a range of P (e.g., P = 1, 5, 10, 50 cm)

**E. Peak flows**

1. Define the time of concentration, and explain why it is important for determining the appropriate IDF duration.
2. Explain why shortening the time of concentration increases the peak discharge for a unit hydrograph.
3. Using the USGS peak flow equations, tabulate peak flows for both urban and rural watersheds for a range of watershed areas.
4. Write and define the terms in the Rational Formula, and then derive the formula assuming that:
   - The time of concentration is one-half of the total hydrograph duration,
   - The hydrograph is triangular, so that the runoff volume is half the product of the duration and the peak discharge
   - The runoff volume is also equal to the product of the rainfall intensity, the time of concentration, and the rational coefficient

**F. Unit hydrographs**

1. Explain how changing the time to peak for a unit hydrograph might affect the time of concentration and the peak discharge.
2. Using precipitation and discharge data of your choice, use regression deconvolution to estimate the unit hydrograph.