Chapter 6
Interaction Between Surface Water and Groundwater

6-1. General

This chapter will provide an overview of the distribution and movement of water between the surface and the subsurface. Practical analytical methods which quantify the interaction between surface water and groundwater are provided. Additionally, an overview on computer modeling of the interaction of groundwater with surface water is presented.

6-2. System Components

a. General. Surface and groundwater systems are in continuous dynamic interaction. In order to properly understand these systems, the important features in each system must be examined. These features are grouped into components referred to as the surface component, the unsaturated zone component, and the groundwater (saturated) component. The flow of water on the surface, and in the unsaturated and saturated zone, is driven by gradients from high to low potentials. Figure 6-1 presents the basic flow components of a surface-groundwater system.

b. Surface water. Surface water is water that flows directly on top of the ground. Surface water includes obvious features of streams, lakes, and reservoirs, and the less obvious features of sheet flow, and runoff from seeps and springs. Runoff across the surface occurs whenever the accumulation from precipitation (either as rain or snow) exceeds the infiltration capacity of the subsurface strata and the evapotranspiration rate, or whenever the rate of groundwater discharge exceeds that which is evapotranspired.

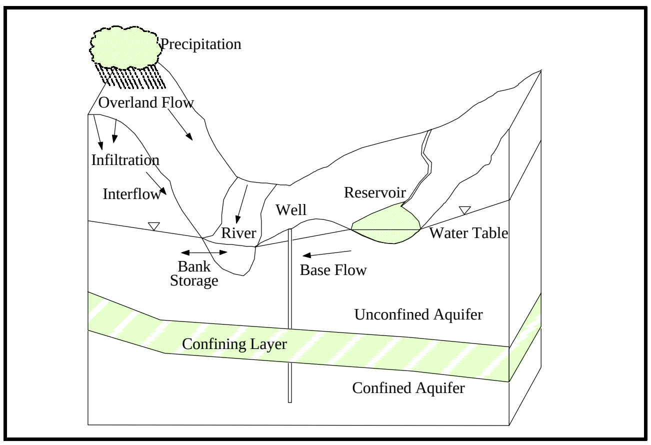

Figure 6-1. Flow components of the surface-groundwater system

c. Subsurface water.

(1) Water infiltrating through the unsaturated zone is from direct precipitation, from overland flow, and from leakage through streambeds. Flow in the unsaturated zone generally is assumed downward in response to gravity. However, poorly permeable strata (for example, a clay layer) can create barriers to downward flow that can limit the amount of water reaching the underlying saturated zone by deflecting flow laterally until it is discharged as evapotranspiration or as seepage to the surface (referred to as interflow).

(2) Groundwater is an important component in the hydrologic cycle as it acts as a large storage reservoir that accepts and releases water from and to the surface. In places where streams flow over permeable strata, the peak flow of a flood is attenuated because of leakage from the stream into the subsurface. Most of this water returns to the stream as the stage decreases, resulting in prolonged flow. This relatively short-term flow of water into and out of the subsurface during a flood event commonly is called bank storage. Additionally, many streams throughout the country have sustained flows during extended dry periods as a result of leakage from groundwater. This contribution to streamflow is called baseflow.

(3) Groundwater flow is complicated by variations in the water-transmitting properties of strata. For example, groundwater can be confined beneath poorly permeable layers of strata (clay or unfractured, dense rock) only to discharge to the surface where land surface cuts through the confining layers or discharge to the surface through fractures that extend through the confining layers (such as a spring). Additionally, pumping of groundwater from permeable sediments near a stream can decrease flow in the stream either by reducing leakage of groundwater to the stream or by inducing leakage from the stream into the subsurface.

6-3. Infiltration

a. General. Infiltration is the process by which water seeps from the surface into the subsurface. The unsaturated zone consists of soil, air, and water (which may be in the form of ice or vapor). The pore space within the soil medium is filled with varying amounts of air and water. Quantifying flow in the unsaturated zone is a much more complicated process than that of the saturated zone primarily because soil properties which control infiltration rates, such as hydraulic conductivity and soil moisture content, tend to change with time.

b. Concepts. Both gravity and moisture potential act to pull water from the surface into the unsaturated zone. Gravity potential is equivalent to elevation (or hydraulic) head. Moisture potential is the negative pressure (or suction) exerted by a soil due to soil-water attraction. The total potential h in unsaturated flow is defined as:

$$h = \Psi(\theta) + Z \qquad (6\text{-}1)$$

where

$\Psi(\theta)$ = moisture potential

Z = gravity potential

Downward flow through the unsaturated zone is controlled by the vertical hydraulic conductivity $K(\theta_v)$ of the soil medium, and moisture potential. The value $K(\theta_v)$ increases as the moisture content increases. At saturation, the vertical hydraulic conductivity $K(\theta_v)$ equals the saturated hydraulic conductivity described by Darcy's Law (Section 2-11), and downward flow is controlled by hydraulic conductivity and elevation head. Moisture potential varies with the moisture content and pore size of the medium. When soil is dry, the moisture potential is typically several orders of magnitude greater than gravity potential. A dry soil will have a higher initial infiltration rate than that of a moist soil, due primarily to free surfaces within the pore space. The pores act as capillary tubes to draw in water, and as they fill, the capillary forces decrease along with the infiltration rate. Figure 6-2 provides an illustration of the relationship between moisture potential, hydraulic conductivity, and water content for a clay sample.

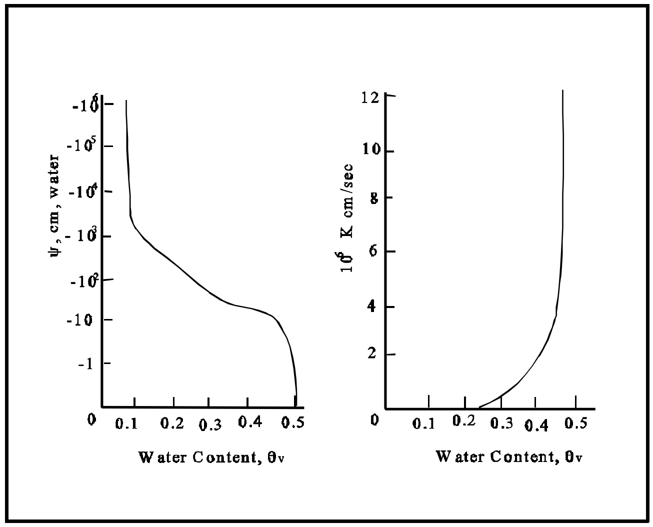

Figure 6-2. Typical relationship between moisture potential (ψ), hydraulic conductivity (*K*) and water content (θ_v) for an identical clay sample

Groundwater flow is described by Darcy's Law:

$$q = -K\,(dh/dl) \qquad (6\text{-}2)$$

where

q = flow rate of groundwater

K = hydraulic conductivity

dh/dl = gradient over which flow occurs

By substituting $\psi + z$ in Equation 6-1 for h in Equation 6-2, infiltration can be described as a function of vertical hydraulic conductivity and moisture potential:

$$q_v = -K_v \frac{\partial(\psi + z)}{\partial z} \qquad (6\text{-}3)$$

As discussed earlier, infiltration and flow in the unsaturated zone are controlled by moisture potential $\psi(\theta)$ as well as hydraulic conductivity $K(\theta_v)$. Thus, any change in the infiltration rate requires a change in moisture content θ. This is described by the one-dimensional Richard's equation:

$$\frac{\partial}{\partial z}\left[K(\theta_v)\left(\frac{\partial\psi}{\partial z} + \frac{\partial z}{\partial z}\right)\right] = \frac{\partial\theta}{\partial t} \qquad (6\text{-}4)$$

The solution to Richard's equation indicates a decrease in moisture potential with cumulative infiltration, and as moisture potential approaches zero, the infiltration rate decreases to a rate equivalent to saturated vertical hydraulic conductivity; i.e.,

$$q \text{ (at saturation)} = -K_v\,(dz/dz) = -K_v \quad (6\text{-}5)$$

Infiltrating moisture from rainfall events tends to move vertically downward as a wave front of saturated soil. Eventually, this wave front reaches the water table and moisture conditions in the soil profile stabilize and return to their pre-rain state.

c. Infiltration capacity curve. The decrease in moisture potential with cumulative infiltration is illustrated by an infiltration capacity curve (Figure 6-3). The initial (or antecedent) infiltration capacity f_0 is typically controlled by the moisture content of the soil. The final (or equilibrium) infiltration capacity f_c is equivalent to the saturated vertical hydraulic conductivity of the soil K_v.

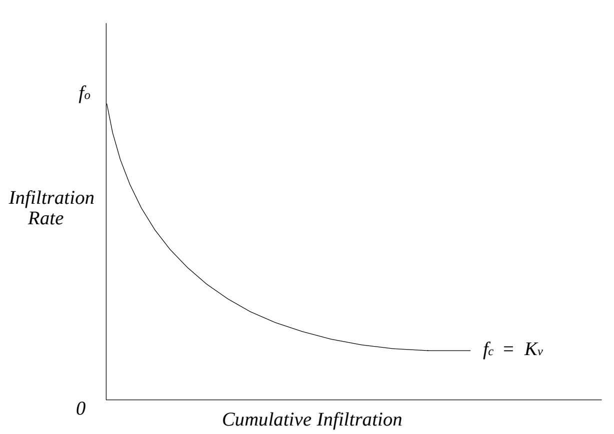

Figure 6-3. Infiltration capacity curve

Other factors affecting soil infiltration include: rainwater chemistry, soil chemistry, organic matter content, and presence of roots and burrowing animals. Field conditions that encourage a high infiltration rate include: low soil moisture, course/porous topsoil, well-vegetated land (inhibited overland flow, lower soil moisture due to transpiration), and land use practices that reduce soil compaction.

6-4. Stream-Aquifer Interaction

Water is transmitted from a basin to a channel system through the following three basic mechanisms (Figure 6-1):

- Overland flow occurs when the rate of precipitation exceeds the infiltration capacity of the local soil.

- Interflow occurs when infiltrating subsurface flow above the water table is diverted towards a channel bed by stratigraphic changes.

- Base flow occurs when the potentiometric surface (or elevation of the water table) proximate to the channel bed exceeds stream elevation.

Seasonal conditions may control the groundwater elevation and thus the direction of flow between the stream and aquifer. When the hydraulic gradient of the aquifer is towards the stream, groundwater discharges to the stream, and the stream is a gaining or effluent stream. When the hydraulic gradient of the aquifer is away from the stream, the stream is losing or influent. The rate of this water loss is a function of the depth of water, the hydraulic gradient towards the groundwater, and the hydraulic conductivity of the underlying alluvium. The channel system can be hydraulically connected to the aquifer, or have a leaking bed through which water can infiltrate to the subsurface. The extent of this interaction depends on physical characteristics of the channel system such as cross section and bed composition. Streams commonly contain a silt layer in their beds which reduces conductance between the stream and the aquifer.

6-5. Interaction Between Lakes and Groundwater

The hydrologic regime of a lake is strongly influenced by the regional groundwater flow system in which it is located. This interaction plays a critical role in evaluating the water budget for the lake. A method of classifying lakes hydrogeologically can be based on the domination of the annual water budget on surface water or groundwater. Lakes dominated by surface water typically have inflow and outflow streams, while seepage lakes are groundwater dominated. Large permanent lakes almost always provide areas of discharge from the local groundwater. The rates of groundwater inflow are controlled by watershed topography and the hydrogeologic environment. Winter (1976) concluded that if the water table is higher than the lake level on all sides of a seepage lake, groundwater will seep into the lake from all sides, including upward seepage through the lake bottom assuming a homogeneous flow system. However, should an aquifer of much higher conductivity underlie the lake, this zone of upward seepage can be eliminated. Three-dimensional numerical analysis of the lake/groundwater interaction system indicated that upward seepage tends to occur around the lake edges, while seepage out of the lake tends to occur in the middle of the lake.

6-6. Analytical Methods

a. General. This section will provide physically based analytical methods for:

(1) Estimating aquifer diffusivity (Section 2-16) from the response of groundwater levels to fluctuations in surface-water levels.

(2) Estimating the groundwater contribution of recharge from a storm event to streamflow.

(3) Using streamflow records to estimate aquifer diffusivity.

(4) Estimating the effects of pumping wells on stream depletion.

b. Baseflow recession. A stream hydrograph describes the flow at a certain point on a river as a function of time. While the overall streamflow shown on a hydrograph gives no indication of its origin, it is possible to break down the hydrograph into components such as overland flow, interflow, and baseflow. After a critical time following a precipitation event when overland flow and interflow are no longer contributing to streamflow, the hydrograph of a stream will typically decay exponentially. Discharge during this decay period is composed entirely of groundwater contributions as the stream drains water from the declining groundwater reservoir. This baseflow recession for a drainage basin is a function of the overall topography, drainage patterns, soils, and geology of the watershed. The slope of baseflow recession is consistent for each watershed and independent of such things as magnitude of the precipitation event or peak flow. When an aquifer contained by a watershed is homogeneous, the hydrograph of a stream at a critical time following a precipitation event (when all discharge to the stream is contributed by groundwater) will decay following an exponential curve. This baseflow recession is described by:

$$Q = Q_0 e^{-kt} \tag{6-6}$$

where

Q = flow at some time t after recession has started

Q_0 = flow at the start of baseflow recession

k = recession constant for the basin

t = time

The value for k, the recession constant, is typically estimated empirically from continuous hydrograph records over an extended period. Rorabaugh (1964) developed a physically based method for estimating the recession constant for the basin based upon aquifer diffusivity (Section 2-17) and basin topography. This allowed for the estimation of baseflow recession in streams with limited continuous data, and also allowed for the estimation of adjacent groundwater properties based upon measured streamflow records.

c. Assumptions. To analyze a stream-aquifer system analytically, many simplifying assumptions need to be made. Assumptions used throughout Sections 6-7, 6-8, and 6-9 include the following:

(1) Darcy's law applies.

(2) The aquifer is homogenous, isotropic, and of uniform thickness.

(3) The rocks beneath the aquifer are impermeable.

(4) The surface-water body fully penetrates the groundwater system, and flow is considered horizontal.

(5) The lateral boundaries of the aquifer are impermeable.

(6) Distances from the stream to groundwater divides or geologic boundaries of flow are for each stream reach; when this distance is termed semi-infinite, this boundary has minimal influence on the analytical solution.

(7) The river is not separated from the aquifer by any confining material.

6-7. Estimating the Transient Effects of Flood Waves on Groundwater Flow

a. Introduction. Accurate estimation of the transmissivity and storage coefficient of an aquifer is critical to the prediction of groundwater flow patterns. If an aquifer is adjacent to a river or surface reservoir which experiences periodic stage fluctuations, it may be possible to calculate these parameters from an analysis of the aquifer response to the fluctuations.

b. Principle. The idealized flow domain is shown in Figure 6-4. The aquifer is represented as a semi-infinite, horizontal confined aquifer of uniform thickness bounded on the left by a reservoir (open boundary). The surface-water body is assumed to completely penetrate the aquifer. The water level in the reservoir fluctuates and causes a corresponding fluctuation in the piezometric head within the aquifer. The one-dimensional flow system is described by the governing equation for linear, non-steady flow in a confined aquifer (see Equation 2-28):

$$\frac{\partial h}{\partial t} = \frac{T}{S}\frac{\partial^2 h}{\partial x^2} \tag{6-7}$$

where

h = rise or fall of piezometric head in the aquifer [L]

x = distance from aquifer-surface body intersection [L]

t = time [T]

S = aquifer storage coefficient

T = aquifer transmissivity [L^2/T]

T/S = aquifer diffusivity [L^2/T]

c. Review of solutions. The solution to the governing equation (Equation 6-7) subject to a fluctuating boundary condition has been presented by several authors (Ferris 1951; Cooper and Rorabaugh 1963; Pinder, Bredehoeft, and Cooper 1969; Hall and Moench 1972). Each of the solutions was derived for the semi-infinite flow domain described above subject to the assumptions listed in Section 6-6. The solutions are derived for confined conditions, although satisfactory results for unconfined conditions will be obtained if:

(1) The location of the computed head is sufficiently far enough from the surface water intersection so that it is unaffected by vertical components of flow.

(2) The range in cyclic fluctuation at the computed location is only a small fraction of the saturated thickness of the formation.

d. Uniform fluctuations. Ferris (1951) observed that wells near bodies of tidal water often exhibit sinusoidal fluctuations of water level in response to periodic changes in tidewater stage. An analogous response was suggested for wells situated adjacent to large surface-water bodies. When the stage of the surface body fluctuates as a simple harmonic motion, a series of sinusoidal waves is propagated outward from the surface body-aquifer intersection through the aquifer. Expressions were developed to determine aquifer diffusivity (T/S) based on the observed values of amplitude, lag, velocity, and wavelength of the sinusoidal changes in groundwater level. If the range of the fluctuation in surface water and an adjacent well is known, aquifer parameters can be derived by:

$$h_{gw} = 2H_{sw}e^{-d\sqrt{\frac{\pi S}{PT}}} \tag{6-8}$$

If the lagtime in occurrence between surface and groundwater maximum or minimum stages is known, then:

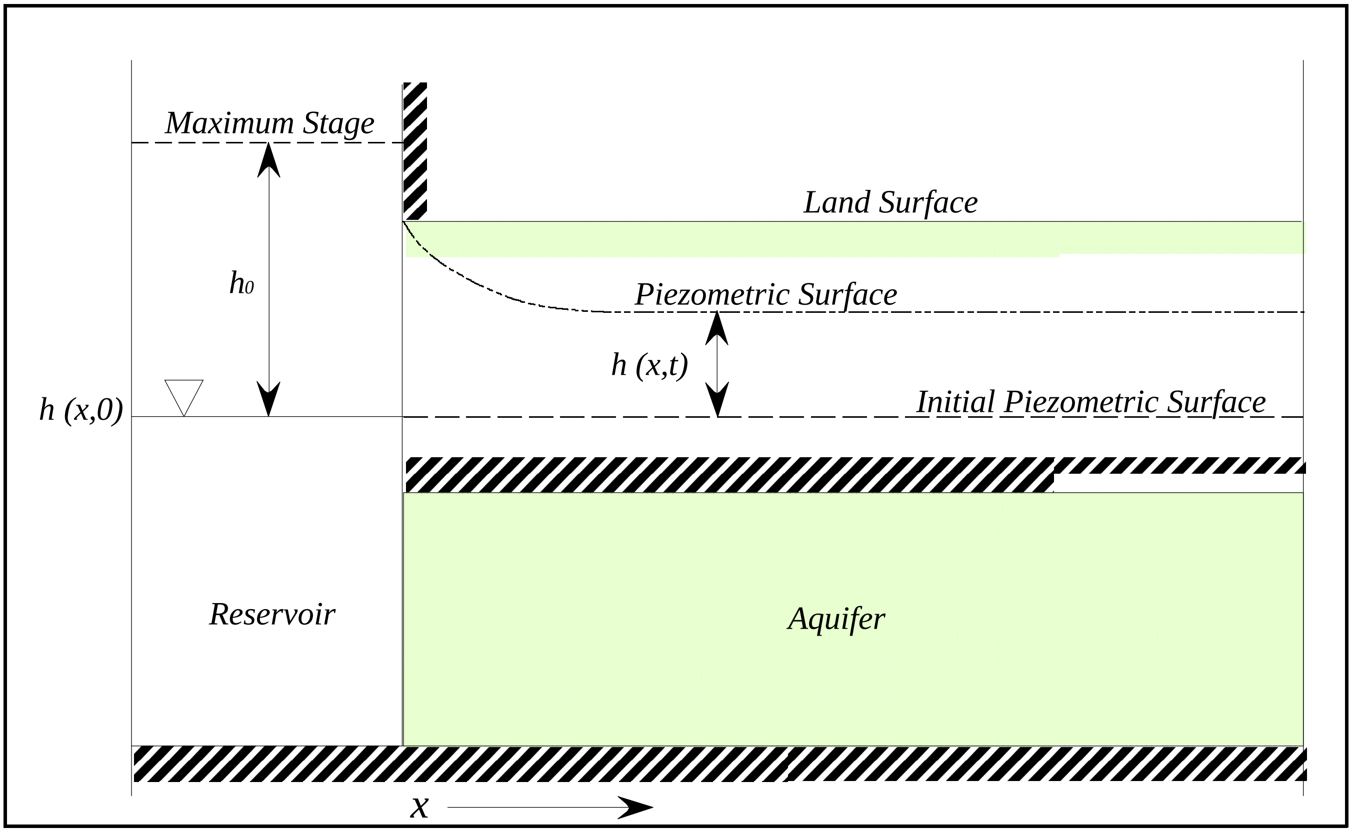

Figure 6-4. Representation of simplified one-dimensional flow as a function of surface-water stage

$$t_{lag} = d\sqrt{\frac{PS}{4\pi T}} \qquad (6\text{-}9)$$

where

h_{gw} = maximum rise in groundwater

H_{sw} = maximum rise in surface-water body

d = distance from well to surface water

S = aquifer storage coefficient

T = aquifer transmissivity

P = period of uniform tide or stage fluctuation

t_{lag} = lag time in occurrence of maximum groundwater stage following the occurrence of a similar surface stage

As shapes of the stage hydrographs for flood waves in surface streams and reservoirs vary widely, a solution of the governing equation satisfying a boundary condition described by a uniform sine wave will generally not approximate the actual domain adequately.

e. Example problem. Sunny Bay has tidal fluctuations every 12 hr with a total tidal change of 3 m. A screened monitoring well 200 m from the shoreline is located within a confined aquifer that is 10m thick. The amplitude of the groundwater change due to the tides is 1 m. *S* was estimated to be 0.001. Estimate the hydraulic conductivity *K* of the aquifer.

Given

h_{gw} = 1 m

H_{sw} = 3 m

d = 200 m

$S = 0.001$

$b = 10$ m

$P = 12$ hr (0.5 day)

Determine K by using Equation 6-8, and substituting Kb for T (Equation 2-9).

$$h_{gw} = 2H_{sw}e^{-d\sqrt{\frac{\pi S}{PT}}}$$

$$1\text{m} = 2(3\ \text{m})\, e^{-200\ \text{m}\sqrt{\frac{\pi(0.001)}{(0.5\ \text{days})(10\ \text{m})(K)}}}$$

$$\ln 0.1667 = \ln(e^{-200\ \text{m}\sqrt{\frac{\pi(0.001)}{(0.5\ \text{days})(10\ \text{m})(K)}}})$$

$$-1.79 = -200\ \text{m}\sqrt{\frac{\pi(0.001)}{(0.5\ \text{days})(10\ \text{m})(K)}}$$

$$0.0090\ \text{m}^{-1} = \sqrt{\frac{0.0006\ \text{m}^{-1}\text{days}^{-1}}{K}}$$

$$0.0001\ \text{m}^{-2} = \frac{0.0006\ \text{m}^{-1}\text{days}^{-1}}{K}$$

$$K = 6\ \text{ft/day}$$

f. Representation of fluctuations by discrete steps. Pinder, Bredehoeft, and Cooper (1969) developed solutions to the governing equation using a discrete approximation of the surface body stage hydrograph. This discrete approach allows the use of a stage hydrograph of any shape (i.e., one not restricted to sinusoidal or uniform asymmetric curves). For each increment in reservoir stage, the head in the adjacent semi-infinite aquifer is given by the solution of Equation 6-7 subject to the boundary and initial conditions

$$h(0,t) = \begin{cases} 0 & when\ t \le 0 \\ \Delta H_m & when\ t > 0 \end{cases} \tag{6-10}$$

$$h(\infty,t) = 0 \tag{6-11}$$

$$h(x,0) = 0 \qquad when\ x \ge 0 \tag{6-12}$$

where

Δh_m = instantaneous rise in surface-water stage at time $t = m\Delta t$ where m is an integer

The solution to the problem is given by:

$$\Delta h_m = \Delta H_m erfc \frac{x}{2\sqrt{(T/S)t}} \tag{6-13}$$

The complementary error function (*erfc*) is unique for each value of x:

$$erfc(x) = \int_x^\infty e^{-t^2}dt \tag{6-14}$$

Values of *erfc* can be found in tables from many sources.

To compute the change in aquifer head (h_p) at the end of any number of stage increments, the change in surface-water stage ΔH_m after each successive increment time Δt must be obtained. The change in groundwater head is given by summing the values of Δh_m computed for each ΔH_m over the period $(p-m)\Delta t$, giving:

$$h_p = \sum_{m=1}^{p} \Delta H_m \left\{ erfc\ \frac{u}{2\sqrt{p-m}} \right\} \tag{6-15}$$

where

h_p = head at a distance x from the reservoir intersection at time $p\Delta t$;

$p\Delta t$ = total time since beginning the period of analysis, where p is the number of time intervals.

$$u = \frac{x}{\sqrt{(T/S)\Delta t}} \tag{6-16}$$

Equation 6-16 can be used to generate type curves for different values of diffusivity. Each set of curves therefore represents the computed change in hydraulic head due to a change in the surface-water stage when selected diffusivities are assumed. The diffusivity of the aquifer is then obtained by choosing from the set of type curves the one which best matches the response observed in adjacent observation wells.

6-8. Estimating Baseflow Contribution from Storm Events to Streamflow

a. Instantaneous recharge.

(1) For flood event scenarios where the precipitation event is of short duration, the assumption of instantaneous recharge can often be made. Rorabaugh (1964) derives an equation which describes base-flow recession at a critical time after an instantaneous uniform increment of recharge ceases to calculate groundwater discharge to a stream:

$$q = 2T(\frac{h_0}{a})(e^{-\pi^2 Tt/4a^2S} + e^{-9\pi^2 Tt/4a^2S} + \ldots) \quad (6\text{-}17)$$

where

q = groundwater discharge [cfs] per foot of stream length (one side) at any time

t = time [days] after recharge ceases [L/T]

h_0 = an instantaneous water table rise, in feet [L]

T = aquifer transmissivity, in ft^2/day [L^2/T]

S = storage coefficient [dimensionless], a common estimate for this value in an alluvial aquifer is 0.20

a = distance from stream to groundwater divide, in feet [L]

(2) This relationship assumes the initial condition that groundwater levels are equal to stream level, and water table fluctuations are small compared to total aquifer thickness.

(3) When $Tt/a^2S > 0.2$, the terms in the series of Equation 6-17 become very small and may be neglected (Rorabaugh 1964):

$$q = 2T(\frac{h_0}{a})e^{-\pi^2 Tt/4a^2S} \quad (6\text{-}18)$$

Conversely, when $Tt/a^2S < 0.2$, Equation 6-17 can be estimated as (Rorabaugh 1964):

$$q = h_0\sqrt{\frac{ST}{t\pi}} \quad (6\text{-}19)$$

(4) Equation 6-19 is for time sufficiently small so that the aquifer response has not reached the groundwater divide, and therefore may be applied to semi-infinite conditions.

(5) The term 'critical time' is defined as the time required in a recession for the profile shape to stabilize, allowing for a straight plot on semi-log graph of streamflow versus time (water levels fall exponentially with time). From stream records, the logarithmic slope of the baseflow recession can be derived after critical time. Critical time (t_c) defines the point on a flow hydrograph where water moving into a stream following a recharge event is derived solely from groundwater (i.e., overland flow in the watershed is no longer a component). Mathematically this term was defined by integrating Equation 6-17 with respect to time:

$$t_c = \frac{0.2a^2S}{T} \quad (6\text{-}20)$$

(6) The total groundwater remaining in storage V from the recharge event, which will eventually be transmitted to a stream at any time after t_c along an entire stream reach l, can be estimated by:

$$V = 2ql(\frac{4a^2S}{\pi^2T}) \quad (6\text{-}21)$$

b. Constant rate of recharge. For the case of a constant rate of recharge (or constant rate of change in river stage), Rorabaugh (1964) derived the following equation:

$$q = CaS\left[1 - \frac{8}{\pi^2}\left(e^{-\pi^2 Tt/4a^2S} + \frac{1}{9}e^{-9\pi^2 Tt/4a^2S} + \frac{1}{25}e^{-25\pi^2 Tt/4a^2S} + \ldots\right)\right] \qquad (6\text{-}22)$$

where

$C = dh/dt$, which is the rate of rise of the water table associated with constant recharge

For values of $Tt/2aS > 2.5$, the exponential terms become insignificant, and flow approaches the steady-state condition:

$$q = CaS \qquad (6\text{-}23)$$

For early time when $Tt/a^2S < 0.2$, effects will not have reached the boundary and the flow is the same as that for a semi-infinite case:

$$q = C(2/\sqrt{\pi})\sqrt{TSt} \qquad (6\text{-}24)$$

c. Constant rate of recharge over a specified time. For the case of constant rate recharge beginning at time t=0, and stopping at time t', Rorabaugh (1964) derived the following equation:

$$q = CaS(8/\pi^2)\left[e^{-\pi^2 Tt'/4a^2S} - e^{-\pi^2 Tt/4a^2S} + \frac{1}{9}e^{-9\pi^2 Tt'/4a^2S} - \frac{1}{9}e^{-9\pi^2 Tt/4a^2S} + \ldots\right] \qquad (6\text{-}25)$$

Analytical methods presented in Section 6-8 assume that precipitation instantaneously recharges the water table. Thus, flow in the unsaturated zone is not addressed. A more accurate estimation of baseflow contribution to streamflow from a storm event can be derived by accounting for soil moisture conditions in the unsaturated zone. However, this accounting requires a numerical complexity that can only be addressed by computer models.

6-9. Estimating Aquifer Diffusivity from Streamflow Records

a. Theory. When critical time is reached, the recession curve on a semilog graph becomes a straight line. Rorabaugh (1960) developed an equation for estimating aquifer diffusivity (*T/S*) from the slope of the water level recession in a stream or observation well after critical time:

$$\frac{T}{S} = \frac{0.933a^2\log(h_1/h_2)}{(t_2 - t_1)} \qquad (6\text{-}26)$$

where

a = distance from stream to groundwater divide [L]

T = aquifer transmissivity [L^2/T]

S = storage coefficient [dimensionless]

h_1 = initial water level [L], at time t_1 [T]

h_2 = water level [L], at time t_2 [T]

This equation is applicable for the condition where recharge is instantaneous and evenly distributed. Assumptions in addition to those stated in Section 6-6 include that the aquifer is thick relative to the change in water level and that the aquifer is wide relative to the thickness. If the base-flow recession curve is evaluated after critical time to determine the time required for streamflow to decline through one log cycle (Δt/log cycle), Equation 6-26 reduces to:

$$\frac{a^2S}{T} = \frac{\Delta t/\text{log cycle}}{0.933} \qquad (6\text{-}27)$$

Combining Equations 6-20 and 6-27 yields a critical time of:

$$t_c = \frac{0.2(\Delta t/\text{log cycle})}{0.933} \qquad (6\text{-}28)$$

These equations also allow for the estimation of baseflow recession if average values of aquifer diffusivity and the distance from the stream to the groundwater divide can be estimated.

b. Methodology. To determine T/a^2S from a recession curve which is declining exponentially with time, the following procedure can be followed (Bevans 1986):

(1) The time that recharge occurred is assumed to be at the point at which streamflow hydrographs reach their peak.

(2) The slope of the baseflow recession curve (days/log cycle) is determined from the recession curve after it becomes a straight line, either by the observed decrease through one log cycle, or by extrapolating the straight line part of the baseflow recession curve through one log cycle.

(3) The slope of the base-flow recession curve is inserted into Equation 6-28 as Δt/log cycle, and the critical time t_c (days) is computed.

(4) The computed critical time is checked against the streamflow hydrograph. The computed critical time needs to be equivalent to the period from the hydrograph peak to the point on the recession curve where the curve becomes a straight line. If computed and observed critical times are the same, then the slope of the baseflow recession curve can be used in Equation 6-27 to compute T/a^2S (1/days). If the computed and observed critical times differ significantly, then extraneous factors probably are affecting the slope of the baseflow recession curve and that particular streamflow record is not appropriate for determining T/a^2S.

(5) T/a^2S values should be determined from several baseflow recession curves, representing different ranges of baseflow rate, and compared to see if T/a^2S is constant. If the values are constant within a narrow range, then T/a^2S can be considered a stream-aquifer constant. Once aquifer diffusivity (T/S) is estimated, the value of transmissivity can be estimated by approximating S from tables; i.e., $S \approx 0.20$ for typical sand aquifers.

c. Example problem.

Given: Extrapolated slope of baseflow recession equals 32 days/log cycle (Figure 6-5). The aquifer is unconfined and consists of shallow sands underlain by bedrock. Assume the aquifer storage coefficient equals 0.20. The stream is located in a valley bounded by bedrock approximately 1,000 m on each side of the stream. Estimate aquifer transmissivity.

Solution:

$$\frac{a^2S}{T} = \frac{\Delta t/\text{log cycle}}{0.933}; \quad \frac{(1{,}000\ \text{m})^2(0.20)}{T}$$

$$= \frac{32\ \text{days}}{0.933}$$

$$T = 5{,}800\ \text{m}^2/\text{day}$$

6-10. Estimating Effects of Pumping Wells on Stream Depletion

a. Assumptions. Jenkins (1968) created a series of dimensionless curves and tables which can be applied to a stream-aquifer system under the following assumptions:

(1) The aquifer is isotropic, homogeneous, and semi-infinite.

(2) Transmissivity remains constant.

(3) The stream is of constant temperature, represents a straight boundary, and fully penetrates the aquifer.

(4) Water is released instantaneously from storage.

(5) The pumping rate is steady during any rate of pumping.

(6) The well is screened through the full saturated thickness of the aquifer.

b. Applications. Computations can be made of:

(1) The rate of stream depletion at any time during the pumping period or the following non-pumping period.

(2) The volume of water induced from the stream during any period, pumping or non-pumping.

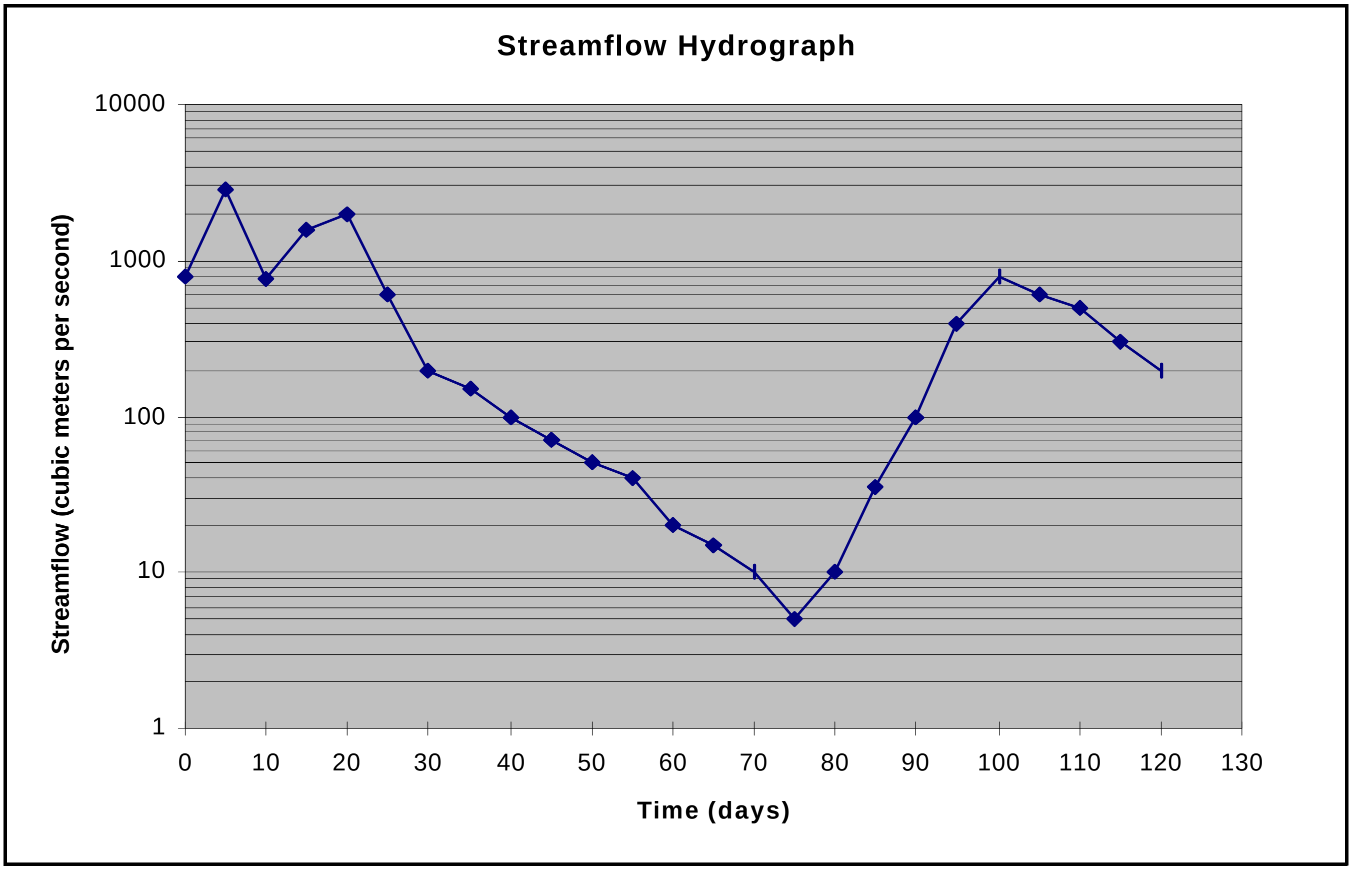

Figure 6-5. Hypothetical streamflow hydrograph

(3) The effects, both in volume and rate of stream depletion, of any pattern of intermittent pumping (Jenkins 1968).

c. Stream depletion factor. Stream depletion means either direct depletion of the stream or reduction of groundwater flow to the stream. In his report, Jenkins introduces a 'stream depletion factor' (sdf) term. If the system meets the above assumptions:

$$sdf = a_w^2 S/T \tag{6-29}$$

where

a_w = distance from the stream to the pumping well

In a complex system, *sdf* can be considered an effective value of $a_w^2 S/T$. This value is dependent upon the integrated effects of irregular impermeable boundaries, stream meanders, areal variation of aquifer properties, distance from the stream, and imperfect connections between the stream and aquifer.

d. Methodology. A simple application of determining the effects of a well, located a given distance *a* from a stream, pumping at a constant rate *Q* for a given time *t* on the volume of stream depletion *v* can be derived from Figure 6-6. First compute the value of *sdf*, then estimate the ratio of *v/Qt* from Figure 6-6. Additionally, the effects of pumping on the rate of stream depletion at a given time *t* after pumping commenced can be easily determined by using Figure 6-6 to determine the ratio *q/Q*. Conversely, the time after pumping begins in which stream depletion will equal a predetermined percentage of the pumping rate can be determined by first computing the ratio of *t/sdf*. Computations for estimating the effects on the river after pumping has stopped, intermittent pumping, and the volume of water induced from the stream during any pumping or non-pumping period can be derived from additional charts and tables in the Jenkins (1968) report.

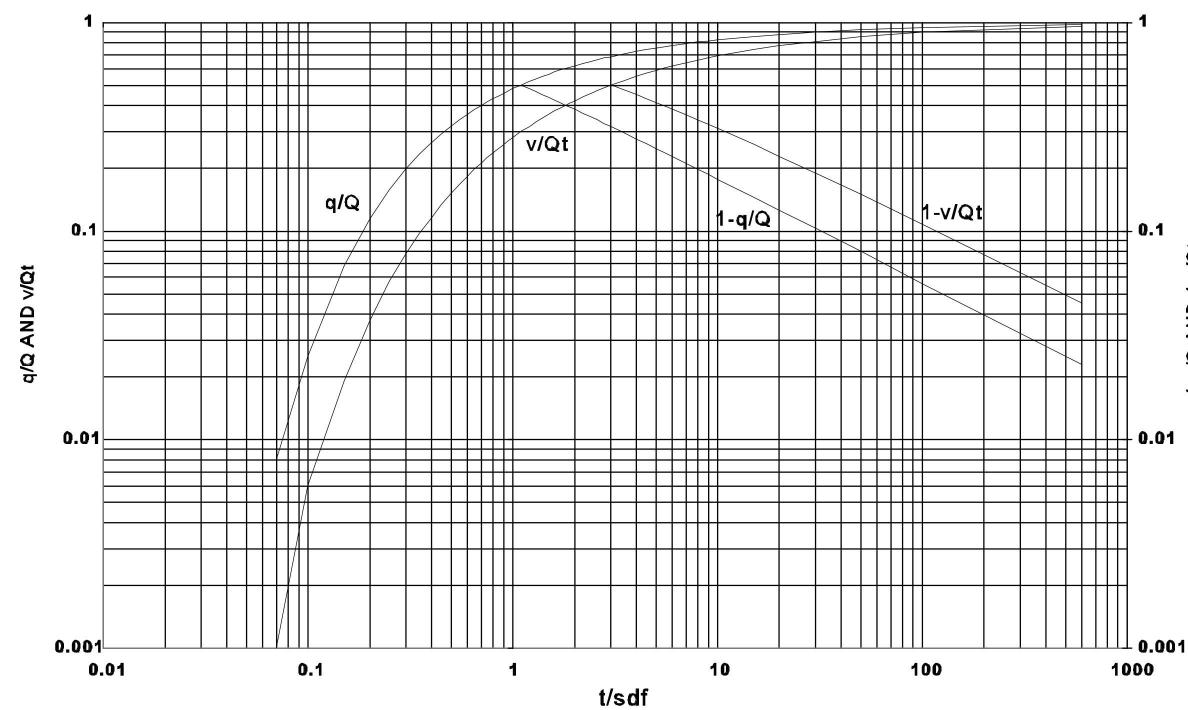

Figure 6-6. Curves to determine rate and volume of stream depletion (Jenkins 1968)

e. Example problem.

(1) The potential influence of aquifer pumping on an effluent stream must be determined. The depth of the stream is 10 m, and depth of the aquifer is 30 m. Assume the stream fully penetrates the aquifer. The hydraulic conductivity of the aquifer is 50 m/day, and specific yield is 0.25. A recently constructed well is located 500 m from the stream (Figure 6-7) and begins pumping at a rate of 1,000 m^3/day.

(2) How much is the total volume of flow in the stream reduced by the pumping well after 2 weeks of pumping?

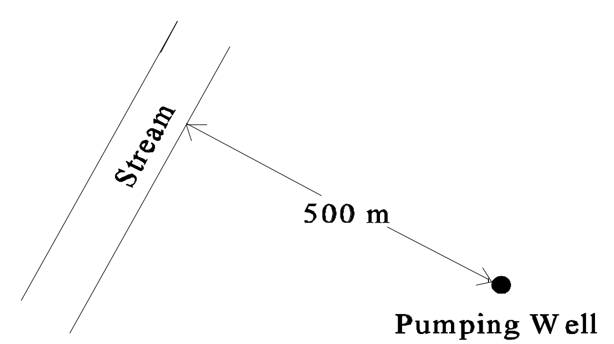

Figure 6-7. Hypothetical stream and pumping well

(a) Given:

Unconfined aquifer
K = 50 m/day
b = 30 m
Q = 1,000 m^3/day
a = 500 m
S = 0.25

(b) Find transmissivity:

$T = Kb$ = (50 m/day)(30 m) = 1,500 m^2/day

(c) Find stream depletion factor (*sdf*):

$sdf = a^2S/T$ = (500 m)^2 (0.25)/(1,500 m^2/day)
= 41.7 days

(d) Estimate ratio of *v/Qt* from Figure 6-6:

t/sdf = 14 days/41.7 days = 0.33

This gives a *v/Qt* value of approximately 0.09

(e) Solve for *v* (total stream depletion)

v = (Q)(t)(0.09) = (1,000 m^3)(14 days)(0.09)
v = 1,260 m^3

(3) What is the rate of stream depletion after 2 weeks of pumping?

(a) The rate of streamflow depletion can be solved by:

t/sdf = 0.3, *q/Q* = 0.22

(b) q = (0.22)(Q). The streamflow is depleted by a rate of 1/5 the pumping rate (220 m^3/day) after 2 weeks of pumping.

(c) Therefore, the total flow in the stream will be depleted by a total of 1,260 m^3 during the first 2 weeks the well is pumping.

6-11. Numerical Modeling of Surface Water and Groundwater Systems

a. General. Although mathematically exact, analytical models generally can be applied only to simple one-dimensional problems because of rigid boundary conditions and simplifying assumptions. However, for many studies, analysis of one-dimensional flow is not adequate. Complex systems do not lend themselves to analytical solutions, particularly if the types of stresses acting on the system change with time. Numerical models allow for the approximation of more complex equations and can be applied to more complicated problems without many of the simplifying assumptions required for analytical solutions. Computer simulation of the interrelationships between surface water and groundwater systems requires the mathematical description of transient effects on potentially complex water table configurations. Ideally, a computer model of the surface-water/groundwater regime should be able to simulate three-dimensional variable-saturated flow including: fluctuations in the stage of the surface-water body, infiltration, flow in the unsaturated zone, and flow in the saturated zone. Additionally, simulation of watershed runoff, surface-water flow routing, and evapotranspiration will allow for completeness. However, this is often a complex task, and no matter how powerful the computer or sophisticated the model, simplifying assumptions are necessary.

b. Modeling stream-aquifer interaction.

(1) General. Numerical models provide the most powerful tools for analysis of the surface-water/groundwater regime. Commonly, interaction between surface water and groundwater is only addressed in the most rudimentary terms. The perspective of the model is of primary importance. In surface-water models, the interaction between surface water and groundwater is often represented as a "black box" source/sink term. Conversely, in groundwater models, surface water is often represented as an infinite source of water, regardless of availability. However, a more precise simulation of the impacts of this interaction can be necessary depending on the objectives of the modeling study.

(2) Theory. From the groundwater perspective, a common simplifying assumption made to ease numerical simulation is that simulation of unsaturated flow is not addressed, and leakage from surface water to an aquifer is assumed to be instantaneous; i.e., no head loss occurs in the unsaturated zone. This assumption is usually reasonable in the common situation where the thickness of the unsaturated zone between the stream and aquifer

is not large. The interaction between surface water and the underlying aquifer can be represented by the partial differential equation of groundwater flow (Equation 2-20):

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right) - W = S_s\frac{\partial h}{\partial t} \tag{6-30}$$

where

W = flow rate per unit volume of water added to or taken from the groundwater system

Most, but not all, interaction between groundwater and surface water is lumped into the "*W*" term.

(3) Specified head and specified flux boundaries. The simplest approach in modeling stream-aquifer interaction is to represent surface water as a specified (constant) head or a specified (constant) flux boundary within the groundwater model grid (Section 5-3). In the case of a specified head boundary, the head at the surface-water location is specified as the elevation of water surface. The flow rate to or from the boundary is computed from heads at adjacent grid points using Darcy's law. This type of boundary does not require a '*W*' term in the partial differential equation of groundwater flow. For the case of the constant, or specified flux boundary, the flow rate is specified in the model grid as a "known" value of recharge or discharge, and the model computes the corresponding head value through the application of Darcy's law. This type of boundary requires the "*W*" term in the partial differential equation of groundwater flow.

A major disadvantage to specified head and flux stream boundary representations is that they do not allow for a lower hydraulic conductivity across the seepage interface, or account for the elevation of the streambed bottom. Thus, leakance from the river continues to increase as the water table drops below the streambed.

(4) Head-dependent flux boundary. A second approach is to represent the stream as a head-dependent flux boundary. A head-dependent flux boundary is a common type of value-dependent boundary discussed in Section 5-3. In this type of boundary condition, the flow which is computed at the stream-aquifer interface is computed as a function of the relative water levels for each stress period. This functional relationship is both included in the "*W*" term of the partial differential equation of groundwater flow and is typically derived from Darcy's law. Thus, the value of groundwater head occurs in the "*W*" term, and in the space derivatives, which can add difficulty to the solution.

(a) Most groundwater flow models incorporate one or more functions built in to handle this functional relationship. For stream-aquifer relationships, this can be represented as:

$$Q = CRIV(h_{riv} - h_{gw}) \tag{6-31}$$

where

Q = flow between the stream and the aquifer

$CRIV$ = streambed conductance

h_{riv} = river stage

h_{gw} = groundwater elevation

The streambed conductance term represents the product of hydraulic conductivity K and cross-sectional area of flow LW divided by the length of the flow path M:

$$CRIV = KLW/M \tag{6-32}$$

Formulation of a streambed conductance term is illustrated in Figure 6-8. If reliable field measurements of stream seepage are available, they may be used to calculate effective conductance. Otherwise, a conductance value must be chosen more or less arbitrarily and adjusted during model calibration. Values for the cross-sectional area of flow can typically be utilized to guide the initial choice of conductance. In general, however, it should be recognized that formulation of a single conductance term to account for three-dimensional flow processes is inherently an empirical exercise, and that adjustment during calibration is almost always required (McDonald and Harbaugh 1988).

(b) An important assumption common in head-dependent flux boundaries of stream-aquifer

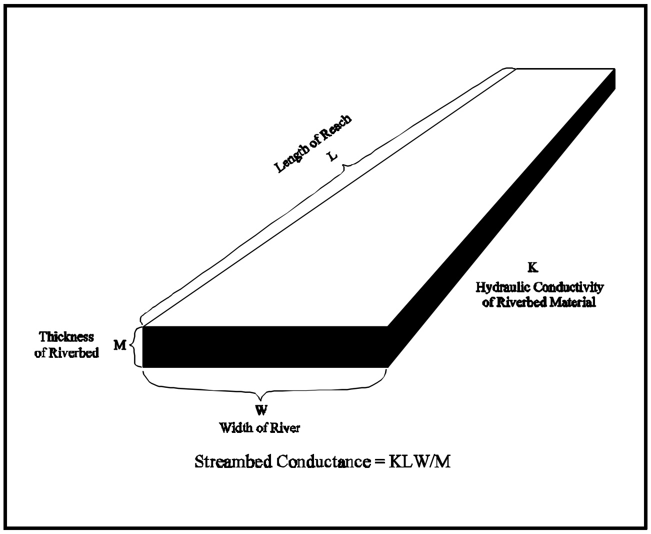

Figure 6-8. Determination of streambed conductance

relationships is that the head differential between the stream and the aquifer is never greater than the sum of stream depth and streambed thickness. In other words, the value of leakage to groundwater does not increase as the groundwater elevation drops below the streambed, and recharge is instantaneous to groundwater. The value of stream bottom elevation is thus also entered into the computational process.

(5) Relationship between cell size and stream width. In groundwater modeling, the smallest unit of homogeneity is represented by the grid cell. Thus, flow between a stream and aquifer is distributed equally over the area of the cell face. For example, if a stream has a width of 50 ft, and the model cell has a width of 300 ft, the same total flow between the stream and the aquifer will be distributed over 36 times the area. Therefore, in situations where the interaction between a stream and an aquifer is of interest, it is important to discretize cell size to approximate river geometry.

(6) Streamflow routing. As discussed previously, the interaction between surface water and groundwater is usually treated as a constant head, constant flow, or as a head-dependent flow boundary. The quantity of surface water in a river, stream, lake, etc., is not accounted for in most groundwater flow simulations. This approach is reasonable for lakes and large rivers where changes in groundwater flow do not appreciably affect the quantity of water in the lakes or rivers. But this approach may not be reasonable for conditions where the amount of surface water is sensitive to changes in groundwater flow. Streamflow routing programs can be used in situations so as not to allow more leakage from streams than there is streamflow. Streamflow routing programs can also allow for the computation of stream stage by inputting an inflow term on the upper reach.

Prudic (1989) developed a simple stream routing computer program, called the "Stream Package," for the U.S. Geological Survey three-dimensional finite-difference flow model MODFLOW. Basic assumptions of the Stream Package are that streamflow entering the modeled area is instantly available to downstream reaches during each time period, leakage between a stream and aquifer is instantaneous, and all stream loss recharges the groundwater system (ET, precipitation, and overland runoff is not accounted for). The Stream Package first computes river stage from Mannings equation (assuming a rectangular channel), then uses the MODFLOW River Package head-dependent flux boundary condition (Equation 6-32) for computing leakage to groundwater flow.

Additionally, the U.S. Geological Survey groundwater flow model MODFLOW has also been coupled to the U.S. Geological Survey unsteady, open-channel flow model BRANCH (Schaffranek et al. 1981). The BRANCH surface flow model simulates flows in networks of open channels by solving the one-dimensional equations of continuity and momentum for river flow. These equations are appropriate for unsteady (changing in time) and nonuniform (changing in location) conditions in the channel. It was developed independent of MODFLOW to simulate flow in rivers without consideration of interaction with the aquifer. BRANCH was modified to function as a module for MODFLOW (Swain and Wexler 1993). Leakance between stream and aquifer is computed through use of a head-dependent flow boundary, Equation 6-31.

c. Modeling interaction between reservoirs (and lakes) and groundwater.

(1) General. Groundwater flow models used to quantify flow between reservoirs (and lakes) and groundwater typically use a specified head to represent the average elevation of the reservoir. However, reservoir levels often show long- and short-term transience in stage and area of inundation. Thus, a model using

specified heads may not provide reliable estimates of groundwater fluxes and reservoir fluctuations over time. As stage increases in reservoirs, a spreading out of the impoundment occurs. Thus, increases in leakage to or from a reservoir are dependent on stage and area of inundation. An algorithm entitled the "Reservoir Package" (Fenske, Leake, and Prudic 1996) was developed for the U.S. Geological Survey three-dimensional finite-difference groundwater flow model MODFLOW to automate the process of specifying head-dependent boundary cells during the simulation. The package eliminates the need to divide the simulation into many stress periods while improving accuracy in simulating changes in groundwater levels resulting from transient reservoir levels. The package is designed for cases where reservoirs are much greater in area than the area represented by individual model cells.

(2) Description. More than one reservoir can be simulated using the Reservoir Package. Figure 6-9 illustrates the specification of the area of potential inundation for two reservoirs. Only those cells specified in the array represented by Figure 6-9 can be activated during model simulations. In cases where areas of higher land-surface elevation separate areas of lower elevations in a reservoir, part of the reservoir may fill before spilling over to an adjacent area. The package can simulate this process by specifying two or more reservoirs in the area of a single reservoir. The area of potential inundation is represented by values of reservoir-bed elevation, layer number, reservoir-bed conductance, and reservoir-bed thickness. Reservoir-bed elevation is the elevation of the land surface within the specified area of potential inundation for each reservoir. Typically, the reservoir-bed elevation at each model cell is equivalent to the average land-surface elevation of the cell.

Reservoir stage is used to determine whether a model cell is activated for each time-step. Whenever stage exceeds land-surface elevation of a cell within the area of potential inundation of a reservoir, the cell is activated. Similarly, whenever reservoir stage is less than the land-surface elevation of a cell, the cell is not activated.

(3) Computation of flow between reservoir and groundwater. Leakage between the reservoir and the underlying aquifer is simulated for each model cell corresponding to the inundated area by multiplying the head difference between the reservoir and the aquifer by the hydraulic conductance. Hydraulic conductance between the reservoir and the aquifer is given by:

$$CRB = \frac{KLW}{M} \qquad (6\text{-}33)$$

where

CRB = reservoir-bed conductance [L^2/T]

K = vertical hydraulic conductivity of the reservoir bed [L/T]

L = cell length [L]

W = cell width [L]

M = reservoir-bed thickness [L]

Values of reservoir-bed conductance and reservoir-bed thickness can be entered into the Reservoir Package as a single parameter or an array.

Reservoir-bed thickness is subtracted from reservoir-bed (or land-surface) elevation to obtain the elevation of the reservoir-bed bottom. The reservoir-bed bottom elevation is used in computing leakage. When the head in the aquifer is above the reservoir-bed bottom, leakage from or to the aquifer is computed by:

$$Q_{res} = CRB(h_{res} - h_{gw}) \qquad (6\text{-}34)$$

where

Q_{res} = leakage from the reservoir [L^3/T]

h_{res} = head in the reservoir [L]

h_{gw} = aquifer head [L]

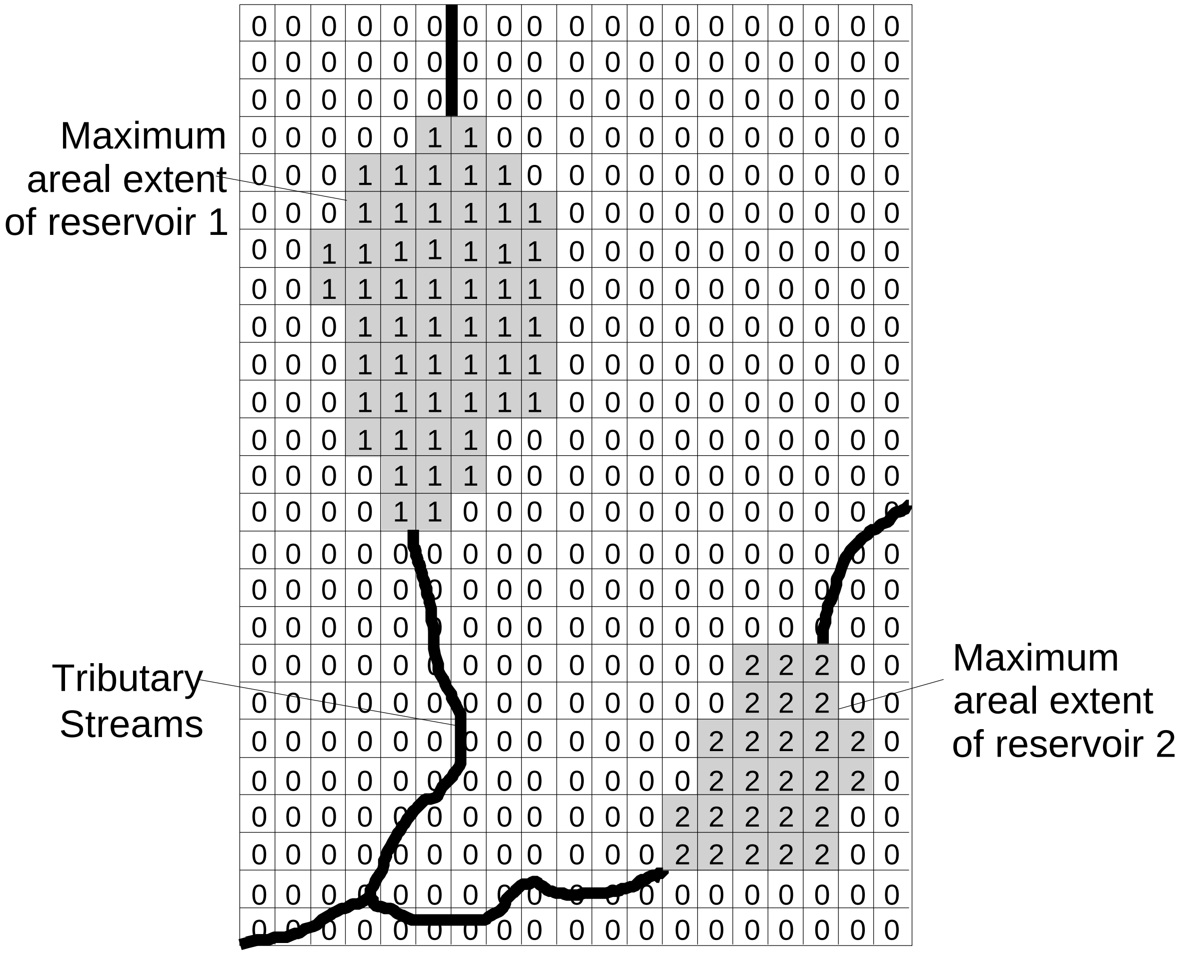

Figure 6-9. Definition of maximum areal extent of reservoir(s)

When the head in the aquifer is less than the elevation of the reservoir-bed bottom, leakage from the reservoir to the groundwater is computed by:

$$Q_{res} = CRB(h_{res} - h_{resbot}) \qquad (6\text{-}35)$$

where

h_{resbot} = elevation of the reservoir-bed bottom [L]

(4) Reservoir package applicability and limitations. Water exchange between surface and subsurface is instantaneous, and it is assumed that there is no significant head loss between the bottom of the reservoir bed and the water table. Water exchange takes place across the horizontal faces of model cells. Thus, bank flow is not directly simulated. The effects of bank flow can be approximated by dividing the reservoir into multiple layers. Changes in reservoir stage are transmitted instantly across the reservoir. Implied in this assumption is that the reservoir has no slope and there is no flow across the reservoir. This assumption may not be valid for large reservoirs. Additionally, head-dependent flow boundaries are

specified for all cells having a land-surface elevation less than the reservoir stage, even if areas of higher land-surface elevations separate areas of lower elevations. This assumption may be unreasonable for reservoirs in which the land surface is uneven and where parts of the reservoir fill before spilling into adjacent lower-lying areas. The package can simulate this process by having two or more reservoirs specified. Neither precipitation on nor evapotranspiration from the reservoir is directly simulated; however, both can be included by adding or subtracting an equivalent volume of water per unit area from the flood stage.