4.1.1

Thursday, September 02, 2010
4:26 PM

A confined aquifer is 18.5m thick. The potentiometric surface elevations at two observation wells 822 m apart are 25.96 and 24.62 m. If the horizontal hydraulic conductivity of the aquifer is 25 m/day, determine the flow rate per unit width of the aquifer, specific discharge, and average linear velocity of the flow assuming steady unidirectional flow.

4.1.1 From Darcy's Law:

$$q = -Kb\frac{dh}{dl} = -(25\,m/day)(18.5\,m)\left(\frac{(24.62\,m - 25.96\,m)}{822\,m}\right) = 0.754\,m^3/day$$

Specific discharge can be found as follows:

$$v = \frac{Q}{A} = \frac{q}{b} = -K\frac{dh}{dl} = -(25\,m/day)\frac{(24.62\,m - 25.96\,m)}{822\,m} = 0.04075\,m/day \cong 4.1\,cm/day$$

The seepage velocity, or average linear velocity is calculated as follows:

$$v_p = \frac{v}{n_e} = \frac{0.04075\,m/day}{0.25} = 0.163\,m/day = 16.3\,cm/day$$

4.1.5

Saturday, September 04, 2010
1:13 PM

Near steady state conditions, explain how the hydraulic gradient changes in the flow direction in

a) Confined aquifer
b) Unconfined aquifer

4.1.5 The potentiometric surface has a linear gradient in a confined aquifer. On the other hand, since the cross sectional area gets smaller in the flow direction, the hydraulic gardient must be greater in order to have the same flow per unit width in an unconfined aquifer.

4.1.9

Saturday, September 04, 2010
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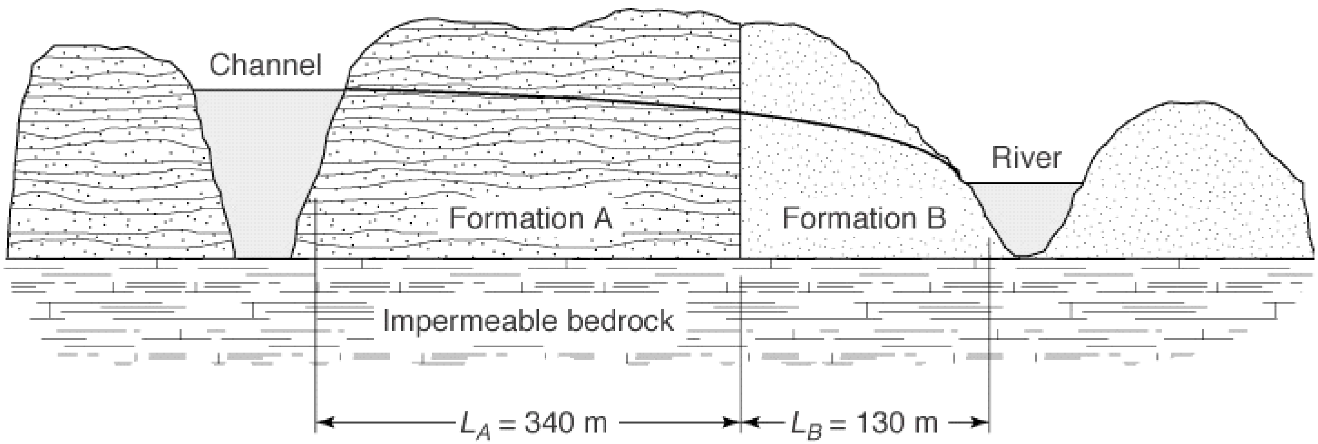
Compute the volume of water that seeps from the channel into the river in the figure above. Water surface elevations relative to bedrock are 13 and 10.5 m. Hydraulic conductivity of A is 5.6 m/day, B is 12.3 m/day.



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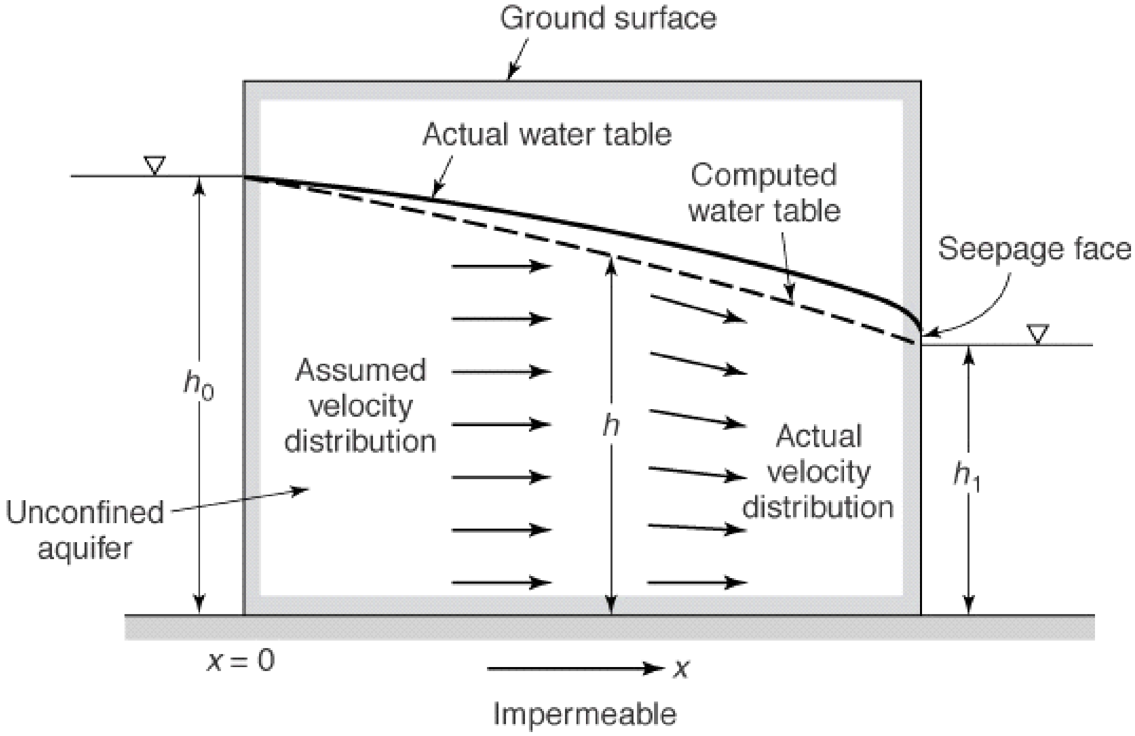
For an unconfined system: $q = K/2x\ (h_0^2 - h^2)$


Ground surface
Actual water table
Computed
water table
Seepage face
Assumed
velocity
distribution
Actual
velocity
distribution
h_0
h
h_1
Unconfined
aquifer
$x = 0$
x
Impermeable

F04_01_02

4.1.9 First we must derive an expression by modifying the Dupuit equation, Eq. 4.1.6, in order to account for multiple formations.

In developing the expression that will account for multiple formations, the subscript s will denote the seperation, or divide between the two formations. Thus, h_s denotes the water table elevation at the seperation. Under steady state conditions, applying the Dupuit equation between the channel and the seperation point, and between the seperation point and the river should yield the same flow rate:

$$q = \frac{K_A}{2L_A}\left(h_C^2 - h_S^2\right) = \frac{K_B}{2L_B}\left(h_S^2 - h_R^2\right)$$

Then, solving for h_S^2 yields:

$$h_S^2 = \frac{\frac{K_A}{2L_A} h_C^2 + \frac{K_B}{2L_B} h_R^2}{\left(\frac{K_A}{2L_A} + \frac{K_B}{2L_B}\right)}$$

Substituting the above expression for h_S^2 into one of the original equations for the flow rate should yield:

$$q = \frac{h_C^2 - h_R^2}{2\left[\frac{L_A}{K_A} + \frac{L_B}{K_B}\right]}$$

Thus, this is the general form of Dupuit equation for the given case. Substituting

$h_C = 13\ m,\quad h_R = 10.5\ m,\quad K_A = 5.6\ m/day,\quad K_B = 12.3\ m/day,\quad L_A = 340\ m,$
$and\quad L_B = 130\ m$:

$$q = \frac{(13\ m)^2 - (10.5\ m)^2}{2\left[\frac{340\ m}{5.6\ m/day} + \frac{130\ m}{12.3\ m/day}\right]} = 0.4121\ m^2/day$$

4.2.2

Thursday, September 09, 2010
2:37 PM

A confined aquifer of 10 m thickness and 16.43 m/day hydraulic conductivity is fully penetrated by pumping well of 0.5 m radius operating at 425 m3/day. Find drawdown under steady state conditions in pumping well and 50 away. Take radius of influence of pumping well as 300 m.

4.2.2 Equation 4.2.6 may be used to determine the drawdown.

$$s_1 - s_2 = \frac{Q}{2\pi T}\ln\frac{r_2}{r_1} \quad \rightarrow \quad s_w - 0 = \frac{425\ m^3/day}{2\pi(16.43\ m/day \times 10\ m)}\ln\frac{300\ m}{0.5\ m} = 2.63\ m$$

So 2.63 m drawdown occurs in the pumping well.

The drawdown 50 m away from the pumping well can be found similarly:

$$s_1 - s_2 = \frac{Q}{2\pi T}\ln\frac{r_2}{r_1} \quad \rightarrow \quad s_1 - 0 = \frac{425\ m^3/day}{2\pi(16.43\ m/day \times 10\ m)}\ln\frac{300\ m}{50\ m} = 0.74\ m$$

4.2.7

Thursday, September 09, 2010
2:37 PM

Pumping 0.75 m radius well, unconfined aquifer 24 m thickness produces 10 L/s. At steady state 30 m observation well, drawdown is 1.6 m. Drawdown in second monitoring well, 60 m away is 1.1m.

a) Hydraulic conductivity
b) Expected drawdown in pumping well
c) Radius of influence

4.2.7 (a) Use Eq. 4.2.9 to calculate the hydraulic conductivity:

$$Q = 10\ L/s = 864\ m^3/day$$

$$r_1 = 30\ m, \qquad h_1 = 24\ m - 1.6\ m = 22.4\ m$$

$$r_2 = 60\ m, \qquad h_2 = 24\ m - 1.1\ m = 22.9\ m$$

$$K = \frac{Q}{\pi\left(h_2^2 - h_1^2\right)}\ln\left(\frac{r_2}{r_1}\right) = \frac{864\ m^3/day}{\pi\left(22.9^2 - 22.4^2\right)}\ln\left(\frac{60\ m}{30\ m}\right) = 8.42\ m/day$$

(b) Using the same equation and either one of the given drawdown data:

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$$8.42\ m/day = \frac{864\ m^3/day}{\pi\left(h_w^{\ 2} - 22.9^2\right)} \ln\left(\frac{0.75\ m}{60\ m}\right) \quad \rightarrow \quad h_w = 19.53\ m$$

So the expected drawdown in the pumping well is (24 m – 19.53 m) = 4.47 m

(c) Using the same equation and either one of the given drawdown data:

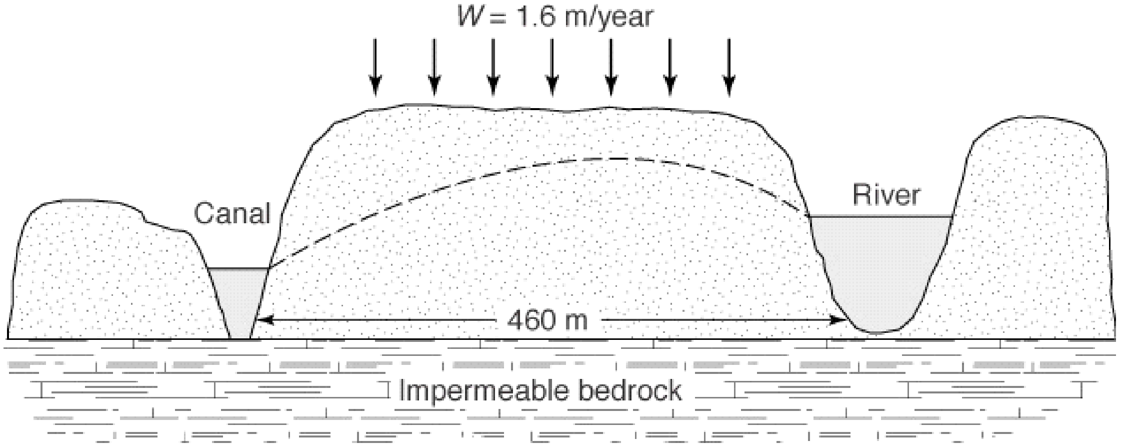
$$8.42\ m/day = \frac{864\ m^3/day}{\pi\left(24^2 - 22.4^2\right)} \ln\left(\frac{r_0}{30\ m}\right) \quad \rightarrow \quad r_0 \cong 291\ m$$

4.1.10

Saturday, September 04, 2010
1:14 PM

Canal paralled to a river, unconfined clean sand and gravel. K=18.5 m/day, infiltration 1.6 m/yr. canal elevation 8.5m water surface, river 10 m. If mound between river and canal becomes contaminated with agricultural chemicals then:

a) Determine discharge into canal and river per kilometer
b) Estimate travel times from water divide to canal and river (porosity=0.35)
c) Propose changes in system to avoid river contamination



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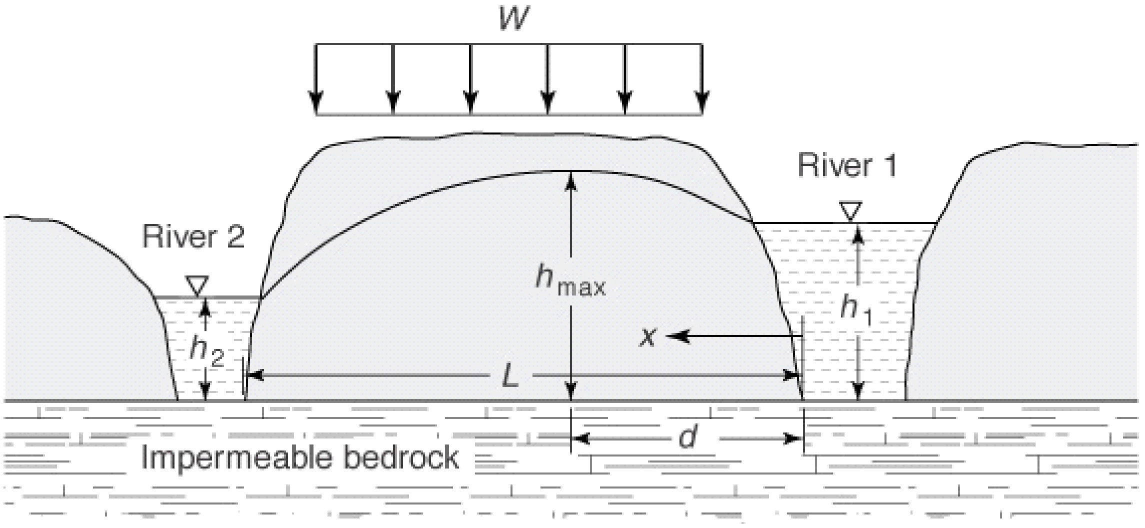
Equation: 4.1.15 $h^2 = (h_1^2 - h_2^2)x + W/K(L-x)x$

Equation 4.1.6 $q_x = K(h_1^2 - h_2^2)/(2L) - W(L/2-x)$

Divide location, $d = L/2 - K/W\ (h_1^2 - h_2^2)/(2L)$

$H2max = h_1^2 - (h_1^2 - h_2^2)d/L + W/K(L-d)d$



F04_01_04

Equation 4.1.16

4.1.10 First we must determine the location of the groundwater divide, where the maximum elevation of water table occurs. Using Eq. 4.1.17

$$d = \frac{L}{2} - \frac{K}{w}\frac{\left(h_1^2 - h_2^2\right)}{2L} = \frac{460\ m}{2} - \frac{18.5\ m/d}{0.0044\ m/d}\frac{\left(10^2 - 8.5^2\right)}{2(460\ m)} = 103.2\ m\ \textit{from the River}$$

$$h_{\max} = \sqrt{h_1^2 - \frac{\left(h_1^2 - h_2^2\right)d}{L} + \frac{w}{K}(L-d)d}$$
$$= \sqrt{10^2 - \frac{\left(10^2 - 8.5^2\right)(103.2)}{460} + \frac{0.0044\,m/day}{18.5\,m/day}(460 - 103.2)(103.2)} = 10.13\ m$$

(a)

$$q_x = \frac{K\left(h_1^2 - h_2^2\right)}{2L} - w\left(\frac{L}{2} - x\right)$$
$$= \frac{(18.5\ m/d)\left(10^2 - 8.5^2\right)}{2(460)} - (0.0044\ m/d)\left(\frac{460}{2} - 0\right) = -0.4540\ (m^3/day)/m$$

So, (0.4540 x 1000 m) = 454.0 m^3/day is the daily discharge from the aquifer into per kilometer of the river.

Similarly, for x = 460 m:

$$q_x = \frac{K\left(h_1^2 - h_2^2\right)}{2L} - w\left(\frac{L}{2} - x\right)$$

$$= \frac{(18.5\ m/d)\left(10^2 - 8.5^2\right)}{2(460)} - (0.0044\ m/d)\left(\frac{460}{2} - 460\right) = 1.5700\ (m^3/day)/m$$

So, (1.5700 x 1000 m) = 1570.0 m^3/day is the daily discharge from the aquifer into per kilometer of the canal.

(b) The average pore velocities can be found by using Darcy's law under the Dupuit-Forchheimer assumption:

$$v_A = \left(\frac{K}{n}\right)\left(\frac{\Delta h}{\Delta x}\right) = \left(\frac{18.5\ m/d}{0.35}\right)\left(\frac{10.13 - 8.5}{460 - 103.2}\right) = 0.2415\ m/day$$

So the travel time from the groundwater divide to the canal is

$$t = \frac{L_A}{v_A} = \frac{460\,m - 103.2\,m}{0.2415\,m/day} = 1477\ days = 4.048\ years$$

Similarly, the travel time from the groundwater divide to the river can be found

$$v_B = \left(\frac{K}{n}\right)\left(\frac{\Delta h}{\Delta x}\right) = \left(\frac{18.5\ m/d}{0.35}\right)\left(\frac{10.13-10}{103.2}\right) = 0.0666\ m/day$$

$$t = \frac{L_A}{v_A} = \frac{103.2m}{0.0666\,m/day} = 1550\ days = 4.247\ years$$

(c) Under given circumstances, groundwater divide is located 103.2 m from the river and the daily discharge into the river is 454 m^3/day. Remembering that the location of groundwater divide is given by:

$$d = \frac{L}{2} - \frac{K}{w}\frac{\left(h_1^2 - h_2^2\right)}{2L}$$

we should be looking for any changes that would shift the water divide towards the river, and even to the right hand side of the river. In fact, mathematically this can be expressed as:

$$d \le 0$$

If the above condition is satisfied, the flow becomes unidirectional (i.e., from the river to the canal only). If we set $d = 0$ and change the parameters in the equation one at a time, the following changes would induce a unidirectional flow:

$$d = \frac{L}{2} - \frac{K}{w}\frac{(h_1^2 - h_2^2)}{2L} = \frac{460\ m}{2} - \frac{K}{0.0044\ m/d}\frac{(10^2 - 8.5^2)}{2(460\ m)} = 0 \rightarrow K = 33.55\ m/day$$

$$d = \frac{460\ m}{2} - \frac{18.5\ m/d}{W}\frac{(10^2 - 8.5^2)}{2(460\ m)} = 0 \rightarrow W = 0.002426\ m/day = 0.8855\ m/day$$

$$d = \frac{460\ m}{2} - \frac{18.5\ m/d}{0.0044\ m/d}\frac{(10^2 - h_C^2)}{2(460\ m)} = 0 \rightarrow h_C = 7.05\ m$$

While increasing the *K* value from 18.5 *m/day* to 33.55 *m/day* does not seem possible, the last two changes (i.e., decreasing the infiltration rate and lowering the water surface elevation in the canal) look more applicable from a practical point of view.

4.4.5

Saturday, September 04, 2010
1:14 PM

4.4.5 The measurements show that the water level is still dropping after 4000 minutes of pumping; therefore, analysis of the test data requires use of the Theis nonequilibrium procedure.

Step 1. Plot the time-drawdown data on log-log graph paper. The drawdown is plotted on the vertical axis and the time since pumping started on the horizontal axis.

Step 2. Superimpose this plot on the type curve sheet so that the plotted points match the type curve. The axes of both graphs must be kept parallel.

Step 3. Select a match point which can be any point in the overlap area of the curve sheets. It is usually taken as the most convenient to select a match point where the coordinates on the type curve are known in advance (e.g., W(u) = 1 and 1/u = 1 or W(u) = 1 and 1/u = 10, etc.). Then determine the value of s and t for this match point:

W(u) = 1 s = 1 *ft* 1/u = 1 t = 2 *min*

Step 4. Determine T using equation (4.4.5)

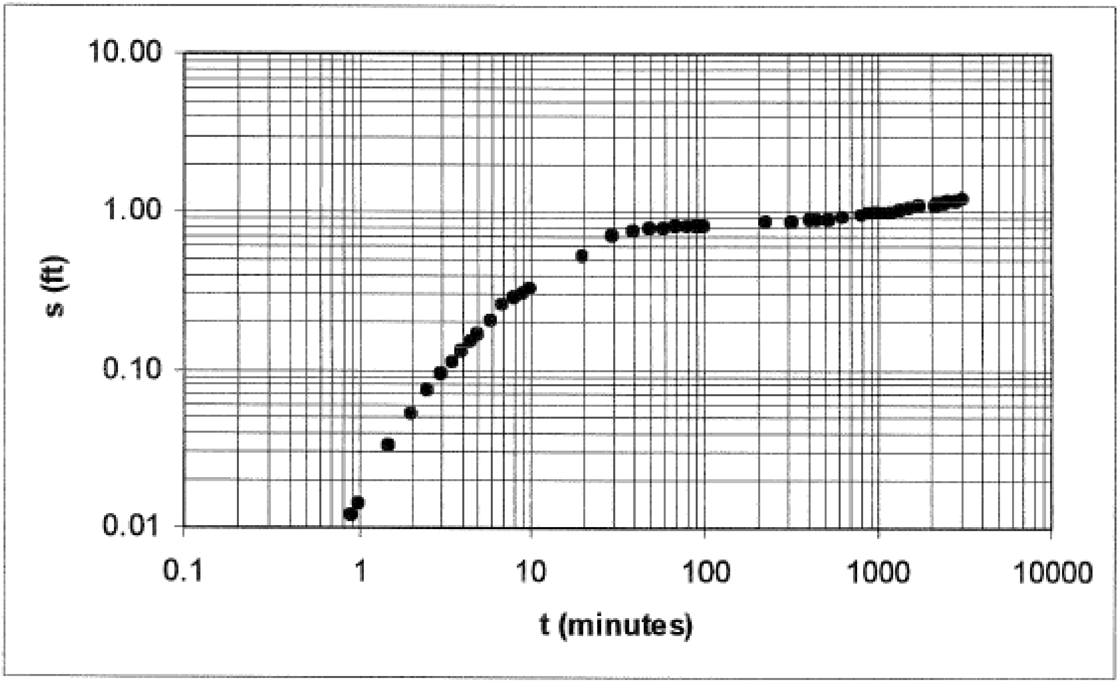
$Q = 500\ gpm = 96250\ ft^3/day$

$$s = \left(\frac{Q}{4\pi T}\right) W(u) \quad \rightarrow \quad T = \left(\frac{Q}{4\pi s}\right) W(u) = \left(\frac{96250\ ft^3/day}{4\pi(1\ ft)}\right)(1.0) = 7659\ ft^2/day$$

Step 5. Determine S using equation (4.4.3)

$$S = \frac{4Ttu}{r^2} = \frac{4(7659\ ft^2/day)\left(\dfrac{2\ \text{min}}{1440\ \text{min}/day}\right)(1.0)}{(200\ ft)^2} = 1.05\times 10^{-3}$$

4.5.1 From the time-drawdown data shown in the following figure, the typical three-phased behaviour for unconfined aquifers can be clearly seen.



Time-drawdown for Problem 4.5.1

The early drawdown versus time data fit best on the type-A curves for $\eta = 0.06$ (see the Type-A curve matching below). The selected match point in the given figure is represented by the following coordinates:

$t = 2.4$ min, $s = 0.40$ ft

$1/u_A = 1.0$, $W(u_A, u_B, \eta) = 1.0$

$$Q = 19.4\ gpm = 3734.5\ ft^3/day$$

From equation 4.5.1

$Q = 19.4\ gpm = 3734.5\ ft^3/day$

From equation 4.5.1,

$$T = \frac{Q}{4\pi s} W(u_A, u_y, \eta) = \frac{(3734.5\ ft^3/day)}{4\pi(0.40\ ft)}(1.0) = 743\ ft^2/day$$

Next, the storativity value can be found using equation 4.5.2:

$$S = \frac{4T u_a t}{r^2} = \frac{4(743\ ft^2/day)(1.0)\left(2.4\ \text{min}\cdot\frac{1day}{1440\ \text{min}}\right)}{(108\ ft)^2} = 0.000425$$

Moving the data curve to the right on the type curve to the best late-time match (for η = 0.06) where s = 0.40 ft (see the following Type-B curve matching) yields

t = 98 min, s = 0.40 ft

$1/u_B = 1$ $W(u_B, \eta) = 1$

Using Eq. 4.5.1 does not change the transmissivity estimate, but using equation 4.5.3

$$S_y = \frac{4T u_y t}{r^2} = \frac{4(743\ ft^2/day)(1.0)\left(98\ \text{min}\cdot\frac{1day}{1440\ \text{min}}\right)}{(108\ ft)^2} = 0.01734$$

The horizontal hydraulic conductivity, K_h, can be found using

$$K_h = \frac{T}{b} = \frac{743\ ft^2/day}{84\ ft} = 8.8\ ft/day$$

Note that, this is very close to the estimate by Kay *et al.* (1989), 8.0 *ft/day*.

Finally, vertical hydraulic conductivity (K_v) of given aquifer can be found using Eq. 4.5.4

$$K_z = \frac{\eta b^2 K_h}{r^2} = \frac{(0.06)(84\ ft)^2(8.8\ ft/day)}{(108\ ft)^2} = 0.32\ ft/day$$

4.6.1

Thursday, September 09, 2010
2:23 PM

4.6.1 The time-drawdown field-data was superimposed on the family type curves for leaky aquifers as shown in the figure below. Comparison shows that the best fit occurs for $r/B = 0.2$. The coordinates of the match point selected are:

$$\frac{1}{u} = 100, \qquad W\left(u, \frac{r}{B}\right) = 1.0$$

$$t = 90\,\text{min}, \qquad s = 2.25\ ft$$

Next we must do the following unit conversions in order to obtain T in units of ft^2/day and permeability of the aquitard in units of ft/day:

$Q = 600\ ft^3/min = 864{,}000\ ft^3/day$

$t = 90\ min = 0.0625$ days

Then the transmissivity and storage coefficient of the confined aquifer are calculated using Equations 4.6.1 and 4.6.2:

$$T = \frac{Q}{4\pi s} W(u, r/B) = \frac{864{,}000\ ft^3/day}{4\pi(2.25\ ft)}(1.0) = 30557.7\ ft^2/day$$

$$S = \frac{4Ttu}{r^2} = \frac{4(30557.7\ ft^2/day)(0.0625\,days)(0.01)}{(160\ ft)^2} = 0.003$$

and permeability of the aquitard is calculated by rearranging Equation 4.6.3:

$$K' = \frac{Tb'(r/B)^2}{r^2} = \frac{(30557.7\ ft^2/day)(14\ ft)(0.2)^2}{(160\ ft)^2} = 0.67\ ft/day$$