CHAPTER 3
METHODOLODY

3.1. Materials and Equipments of Research

The materials will be used in the research consist of:

1. Topographic map.
2. Geological map.
3. Hydrogeological map.
4. Land Use map.
5. Soil map.

The equipments that be used in the research as follow:

1. Software: Visual Modflow version 3.1, Aquitest, REF-ET, Rockworks, surfer and Arc-View.
2. One set computer and printer.
3. GPS (Global Position System).
4. Sample place for water analyses.

3.2. Stratigraphic Analysis

In this method, all lithologic log data are carefully examined and stored in a computer input device, after which they are used to generate a series of horizontal sections, spaced at a small uniform interval using Rockworks software. Different lithologies are separated in these sections based on some criterion relevant to the study concerned. This set of horizontal sections is then used to prepare vertical stratigraphic sections along any given line.

The result from stratigaphic analysis will be used to determine the boundary of aquifers. After decided the boundary of each aquifer from well log data, the correlation for aquifer spreading in the horizontal and vertical directions will be made. The correlations of aquifer are interpolated using result from geostatistical estimation.

Isopach maps are generated for each aquifer and aquitard from kriging estimation. Finally, the conceptual model of groundwater flow modeling is constructed using this relevant information.

3.3. Groundwater Flow

3.3.1. Darcy's law

The movement of groundwater is well understood by hydraulic principles reported in 1856 by Henri Darcy, who investigated the flow of water through beds of permeable sand. Darcy advanced one of the most important laws in hydrology that the flow rate through porous media is proportional to the head loss and inversely proportional to the length of the flow path.

Figure 3.1. illustrates the experimental set up for determining head loss through a sand column, with piezometers located a distance L apart. The Bernoulli Equation can express the total energy for this system as:

$$\frac{p_1}{\gamma} + \frac{v_1^2}{2g} + z_1 = \frac{p_2}{\gamma} + \frac{v_2^2}{2g} + z_2 + h_1 \qquad (3.1)$$

Where p = pressure; γ = specific weight of water; v = velocity; z = elevation; h_1 = head loss.

Because velocities are very small in porous media, velocity heads may be neglected, allowing head loss to be expressed as:

$$h_1 = [(p_1/\gamma) + z_1]-[(p_2/\gamma) + z_2] \qquad (3.2)$$

It follows that the head loss is independent of the inclination of the column. Darcy related flow rate to head loss and length of column though a proportionality constant referred to as K, the hydraulic conductivity, and a measure of the ability of the porous media to transmit water. Darcy's Law can be stated thus:

$$v = -(Q/A) = -K\,(dh/dL) \qquad (3.3)$$

The negative sign indicated that flow of water is in the direction of decreasing head. The Darcy velocity that result from above is an average discharge velocity, v, through the entire cross section of the column. The actual flow is limited to the pore space only, so that the seepage velocity v_s is equal to the Darcy velocity divided by porosity:

$$v_s = Q/nA \qquad (3.4)$$

Thus actual seepage velocities are usually much higher (by a factor of 3 or 4) than the Darcy velocities. Seepage velocity is used later in the text for all transport calculations.

It should be pointed out that Darcy's law applies to laminar flow in porous media, and experiments indicate that Darcy's law is valid for Reynolds number ($R = \rho vd/\mu$) less than 1 and perhaps as high as 10.

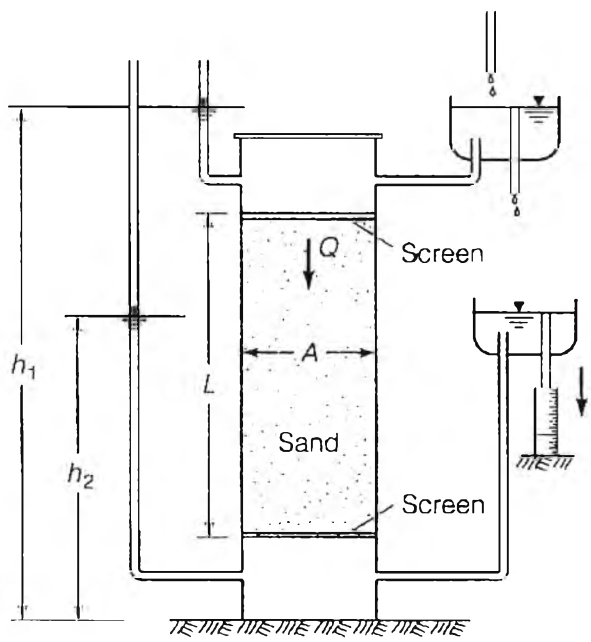


Figure 3.1. Darcy's experimental setup (From David A. Chin, 2000).

From the Darcy's equation we can derive equation for steady flow in a confined aquifer. The quantity of flow per unit width, Q, can be determined as follow:

$$Q = Kb\frac{dh}{dl} \tag{3.5}$$

where,

Q is the flow per unit width (L^2/T)

K is the hydraulic conductivity (L/T)

b is the aquifer thickness (L)

$\frac{dh}{dl}$ is the slope of potentiomentric surface (dimensionless)

The equation for steady flow in an unconfined aquifer can be expressed as follow:

$$Q = \frac{1}{2}K\left(\frac{h_1^2 - h_2^2}{L}\right) \tag{3.6}$$

where,

Q is the flow per unit width (L^2/T)

K is the hydraulic conductivity (L/T)

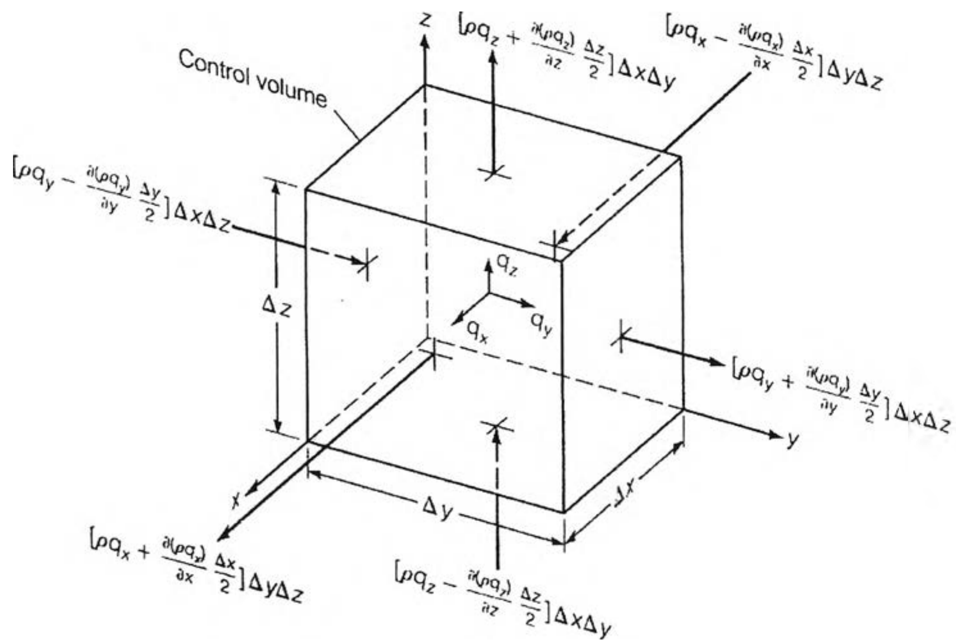
h_1 is the head at the origin (L)

h_2 is the head at L (L)

L is the flow length (L)

3.3.2. Equation of Groundwater Flow

Consider the control volume shown in Figure 3.2. The net influx of fluid mass into the control volume is given by:



$$\begin{aligned}\text{Net mass inflow} = & \left[(\rho q_x) - \frac{\partial(\rho q_x)}{\partial x}\frac{\Delta x}{2}\right]\Delta y \Delta z - \left[(\rho q_x) + \frac{\partial(\rho q_x)}{\partial x}\frac{\Delta x}{2}\right]\Delta y \Delta z \\ & + \left[(\rho q_y) - \frac{\partial(\rho q_y)}{\partial y}\frac{\Delta y}{2}\right]\Delta x \Delta z - \left[(\rho q_y) + \frac{\partial(\rho q_y)}{\partial y}\frac{\Delta y}{2}\right]\Delta x \Delta z \\ & + \left[(\rho q_z) - \frac{\partial(\rho q_z)}{\partial z}\frac{\Delta z}{2}\right]\Delta x \Delta y - \left[(\rho q_z) + \frac{\partial(\rho q_z)}{\partial z}\frac{\Delta z}{2}\right]\Delta x \Delta v \quad (3.7)\end{aligned}$$

Figure 3.2. The net influx of fluid mass into the control volume (from David A. Chin, 2000).

Where ρ is the density of the fluid. Combining terms in equation 3.7 and simplifying leads to:

$$\text{Net mass inflow} = -\left[\frac{\partial(\rho q_x)}{\partial x} + \frac{\partial(\rho q_y)}{\partial y} + \frac{\partial(p q_z)}{dz}\right]\Delta x \Delta y \Delta z \qquad (3.8)$$

In accordance with the law of conservation of mass, the net mass inflow into the control volume is equal to rate of change of mass within the control volume, which is given by:

$$\text{Rate of change of mass} = \frac{\partial(n\rho)}{\partial t}\Delta x \Delta y \Delta z \qquad (3.9)$$

Where n is the porosity of the porous medium, and t is the time. Combining equations 3.8 and 3.9 yield the following relation:

$$-\left[\frac{\partial(\rho q_x)}{\partial x}+\frac{\partial(\rho q_y)}{\partial y}+\frac{\partial(\rho q_z)}{\partial x}\right]=\frac{\partial(n\rho)}{\partial t} \tag{3.10}$$

This equation can be further reduced for the cases where the density, ρ, is independent of space and time. Under this condition, equation 3.10 reduces to:

$$-\left(\frac{\partial q_x}{\partial x}+\frac{\partial q_x}{\partial y}+\frac{\partial q_z}{\partial z}\right)=\frac{\partial n}{\partial t} \tag{3.11}$$

The specific storage, Ss, of a porous medium is defined as the volume of water released from storage per unit volume of the porous medium per unit decline in piezometric head. Considering the control volume shown in Figure 3.2, the time rate of change of fluid volume within the control volume can be expressed in terms of the specific storage, Ss, by the following relation:

$$\frac{\partial n}{\partial t}\Delta x\,\Delta y\,\Delta z = Ss\,\frac{\partial \phi}{\partial t}\Delta x\,\Delta y\,\Delta z \tag{3.12}$$

Where ϕ is the piezometric head, defined as $p/\gamma + z$. Equation 3.12 simplifies into:

$$\frac{\partial n}{\partial t} = Ss\,\frac{\partial \phi}{\partial t} \tag{3.13}$$

And combining equation 3.11 and 3.13 yields:

$$-\left(\frac{\partial q_x}{\partial x}+\frac{\partial q_y}{\partial y}+\frac{\partial q_z}{\partial z}\right)=Ss\,\frac{\partial \phi}{\partial t} \tag{3.14}$$

According to Darcy's law, the components of the specific discharge vector, q_x, q_y, and q_z can be written in terms of the piezometric head gradients, where:

$$q_x = -K_{xx}\frac{\delta\phi}{\delta x} \quad ; q_y = -K_{yy}\frac{\delta\phi}{\delta y} \quad ; q_z = -K_{zz}\frac{\delta\phi}{\delta z} \qquad (3.15)$$

And it is assumed that the coordinate axes are in the directions of the principal axes of the hydraulic conductivity. Combining equation 3.14 and 3.15 lead to:

$$\frac{\partial}{\partial x}\left(Kxx\frac{\partial\phi}{\partial x}\right)+\frac{\partial}{\partial y}\left(Kyy\frac{\partial\phi}{\partial y}\right)+\frac{\partial}{\partial z}\left(Kzz\frac{\partial\phi}{\partial z}\right)=Ss\frac{\partial\phi}{\partial t} \qquad (3.16)$$

This general equation is applicable to both isotropic and anisotropic formation. In cases where the aquifer is homogeneous and anisotropic, equation 3.16 becomes:

$$Kxx\frac{\partial^2\phi}{\partial x2}+Kyy\frac{\partial^2\phi}{\partial y2}+Kzz\frac{\partial^2\phi}{\partial z2}=Ss\frac{\partial\phi}{\partial t} \quad \text{Homogeneous, anisotropic} \qquad (3.17)$$

In cases where the hydraulic conductivity is isotropic, the hydraulic conductivity, K, is independent of the coordinate direction, which means that:

$$K_{xx} = K_{yy} = K_{zz} = K \qquad (3.18)$$

And equation 3.17 can be written as:

$$\frac{\partial^2\phi}{\partial x^2}+\frac{\partial^2\phi}{\partial y^2}+\frac{\partial^2\phi}{\partial z^2}=\frac{Ss}{K}\frac{\partial\phi}{\partial t} \quad \text{Homogeneous, isotropic} \qquad (3.19)$$

Or in the more convenient vector forms as:

$$\nabla^2 \phi = \frac{Ss}{K} \frac{\partial \phi}{\partial t} \quad \text{Homogeneous, isotropic} \tag{3.20}$$

Where ∇^2 () is the Laplacian operator defined by:

$$\nabla^2 f = \frac{\partial^2 f}{\partial x^2} + \frac{\partial^2 f}{\partial y^2} + \frac{\partial^2 f}{\partial y^2} \tag{3.21}$$

Where f is a scalar function. In cases where there is significant radial symmetry, it is convenient to use cylindrical coordinates, and the Laplacian in Equation 3.21 is given by:

$$\nabla^2 f = \frac{\partial^2 f}{\partial r^2} + \frac{1}{r} \frac{\partial f}{\partial r} + \frac{1}{r^2} \frac{\partial^2 f}{\partial \theta^2} + \frac{\partial^2 f}{\partial z^2} \tag{3.22}$$

3.4. Type of Aquifer and Aquifer Properties

3.4.1. Type of aquifer

An aquifer is a geologic formation containing water that can be withdrawn at significant amounts. Aquifers are classified as either unconfined or confined. Unconfined aquifers are open to the atmosphere and also called phreatic aquifers. Figure 3.3. Illustrates different types of aquifer in groundwater system.

In confined aquifers, water in the saturated zone is bounded by either semi-impervious or impervious formations such as aquicludes and aquifuges. Aquicludes is a geologic formation containing water but are incapable of transmitting in significant quantities. Aquifuges is a geologic formation neither contains nor transmits water.

3.4.2. Aquifer Properties

A. Porosity

Porosity (n) is defined as the volume of the pores of a rock or soil sample (Vp) divided by the total volume (Vt) of both pores and solid material.

When a rock is first formed by precipitation, cooling from an igneous melt, indurations from loose sediment, or when a soil is first formed by weathering or rock materials and possibly subsequent biological action, the new entity will contain a certain inherent porosity known as primary porosity. This porosity may later be reduced by cementation from precipitates from circulating groundwater, or from compaction accompanying burial by later sediments. However, fractures or solution cavities formed in the rock, or root tubes or animal burrows in soils may later form and are known as secondary porosity. If all the pores in a rock are not connected, only a certain fraction of the pores will allow the passage of water, and this fraction is known as the effective porosity.

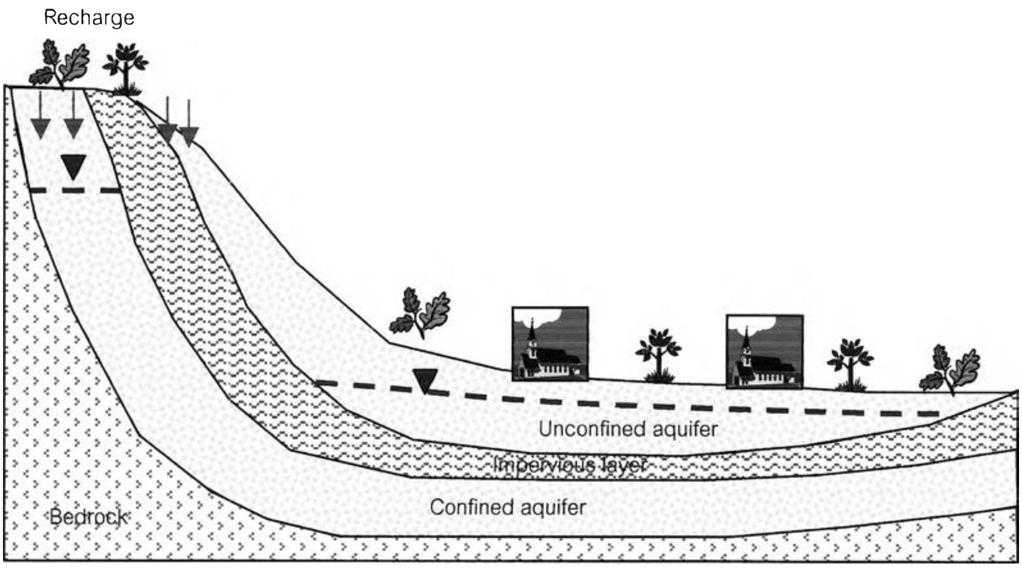


Figure 3.3. Type of aquifer in groundwater system.

Table 3.1. Classification and representative of hydrologic properties in unconsolidated formation.

Material	Particle Size (mm)	Porosity	Specific Yield	Hydraulic conductivity (m/d)
Very coarse gravel	32.0 – 64.0	-	-	-
Coarse gravel	16.0 – 32.0	0.25 – 0.40	0.10 – 0.25	860 – 8,600
Medium gravel	8.0 – 16.0	-	0.15 – 0.45	20 – 1,000
Fine gravel	4.0 – 8.0	0.25 – 0.40	0.15 – 0.40	-
Very fine gravel	2.0 - 4.0	-	-	-

Very coarse sand	1.0 – 2.0	-	-	-
Coarse sand	0.5 – 1.0	0.20 – 0.50	0.15 – 0.45	0.1 – 860
Medium sand	0.25 – 0.5	0.30 – 0.40	0.15 – 0.45	0.1 – 50
Fine sand	0.125 – 0.25	0.25 – 0.55	0.01 – 0.45	0.01 – 40
Very fine sand	0.062 – 0.125	-	-	-
Silt	0.004 – 0.062	0.35 – 0.70	0.01 – 0.40	10^{-5} – 10^{-2}
Clay	< 0.004	0.35 – 0.70	0.01 – 0.20	< 10^{-2}

Source: Morris and Johnson (1976)

B. Storage coefficient

In a unit volume of saturated porous matrix, the volume of water that will be taken into storage under a unit increase head, or the volume that will be released under a unit decrease in head is called specific storage, is shown as:

$$Ss = \rho g (\alpha + n\beta) \quad (3.23)$$

Where α is aquifer compressibility, ρ is fluid density, g is gravitational acceleration, n is porosity, and β is water compressibility. This unit has the dimension of 1/L and is quite small, usually 0.0001 or less.

The storage coefficient of an aquifer, or simply, the storativity, S, is given as:

$$S = S_s b \quad (3.24)$$

Where b is the saturated thickness of the aquifer. Storativity is defined as the volume of water per unit aquifer surface area taken into or released from storage per unit increase or decrease in head, respectively. It is dimensionless quantity. In confined aquifer the value of storativity ranges from 0.005 to 0.00005.

In unconfined porous media, that is, where there is no overlying confining cover, storage of water in its upper part is defined as specific yield, $S\gamma$. This is the ratio of the volume of water that drains from a saturated porous matrix under influence of gravity to the total volume of the matrix per unit drop in the water table. Specific yield is normally

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much greater than specific storage, as water released from elastic storage leaves the pores still saturated. Specific yield is often in the range of 0.2 to 0.3, or three to four orders of magnitude greater than elastic storage.

C. Intrinsic permeability

The intrinsic permeability is simply a function or the average pore size of the medium and is related to this property as follow:

$$k = Cd^2 \quad (3.25)$$

Where d is the average pore diameter, and C is an empirical constant, which depends upon packing, sorting and other factors.

Intrinsic permeability is commonly measured in terms of the Darcy, the millidarcy is the commonly used unit. One darcy is equal to 9.87 x 10^{-9} cm.

D. Hydraulic conductivity of saturated media

Hydraulic conductivity is the volume of liquid flowing perpendicular to a unit area of porous medium per unit time under the influence of a hydraulic gradient of unity.

$$K = k\rho g/\mu \quad (3.26)$$

This parameter has the dimension of velocity, generally cm/sec or m/day, and is a second-order tensor quantity.

E. Transmisivity

The definition of transmisivity is the volume of water per unit time passing through a unit width area of aquifer perpendicular to flow integrated over the thickness of the aquifer.

Transmisivity is simply the product of hydraulic conductivity and saturated thickness of the aquifer:

$$T = Kb \qquad (3.27)$$

And has the dimension of L^2/T.

3.4.3. Anisotropy and heterogeneity

The porous media is isotropic, it means that we assumed the permeability and hence, the hydraulic conductivity and transmisivity, are equal in all directions at any point in the porous medium. If the parameters differ in value directionally at a point, the medium is then said to be anisotropic.

If the condition of directional equality of properties is the same from point to point anywhere in the medium, the medium is termed homogeneous. If the condition of either isotropy or anisotropy varies from point to point, the medium is then said to be heterogeneous.

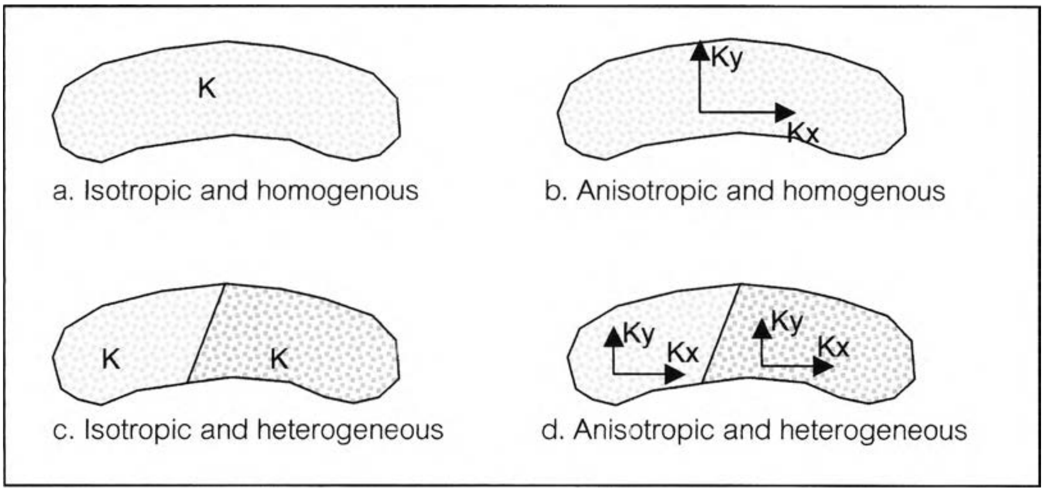

Figure 3.4. Summary of possible combination of isotropy, anisotropy, homogeneity and heterogeneity.

3.5. Groundwater Potential

Basically, potential of groundwater is measured by quantity and quality of groundwater. We can divide the quantity of groundwater into two majors reserves; static reserve and dynamic reserve. The static reserve is the amount of water that stored in the aquifer in the one time. The dynamic reserve is the amount of water that flowing to the aquifer in the one time. The calculation of groundwater reserve includes the maximum of water that can be pumped without effect to the environmental. This concept is called safe yield concept.

The quality of groundwater is determined by analyses of physical, biological and chemical behaviors of groundwater samples from the field in the laboratory. The result of analyses will be compared with the Standard Values for Drinking Purpose. Based on that information we can define the area where the quality of water is not suitable for drinking water and the water must be treated before use.

3.6. Groundwater Modeling

The modeling of groundwater flow in the study area aims to analyses the groundwater system in that area. The analysis takes into account of time and spatial responses using Visual Modflow Version 3.1. Groundwater flow modeling is conducted for both steady stated and transient simulation. The result from steady stated condition will be used as initial condition for transient simulation. To check the accuracy of the model, the continuously measurement groundwater level data from the field are used in the calibration. After all simulations are calibrated, the final model is obtained and will be used for simulations with assuming different pumping scenarios. The flowchart of research methodology is shown in Figure 3.5.

3.6.1. Conceptual models

A key step in the modeling process is to formulate a conceptual model of the system being modeled. A conceptual model is a pictorial representation of the groundwater flow and transport system, frequently in the form of a block diagram or a cross section (Anderson and Woessner, 1992). The nature of the conceptual model will determine the dimensions of the numerical model and the design of the grid.

The purpose of building a conceptual model is to simplify the field problem and make it more amenable to modeling. Even though, a complete reconstruction of this system in a ground water flow model is not feasible, however, a conceptual model of the system can be constructed by identifying the pertinent hydrologic feature of the geologic framework. Formulating a conceptual model for flow and/or transport includes

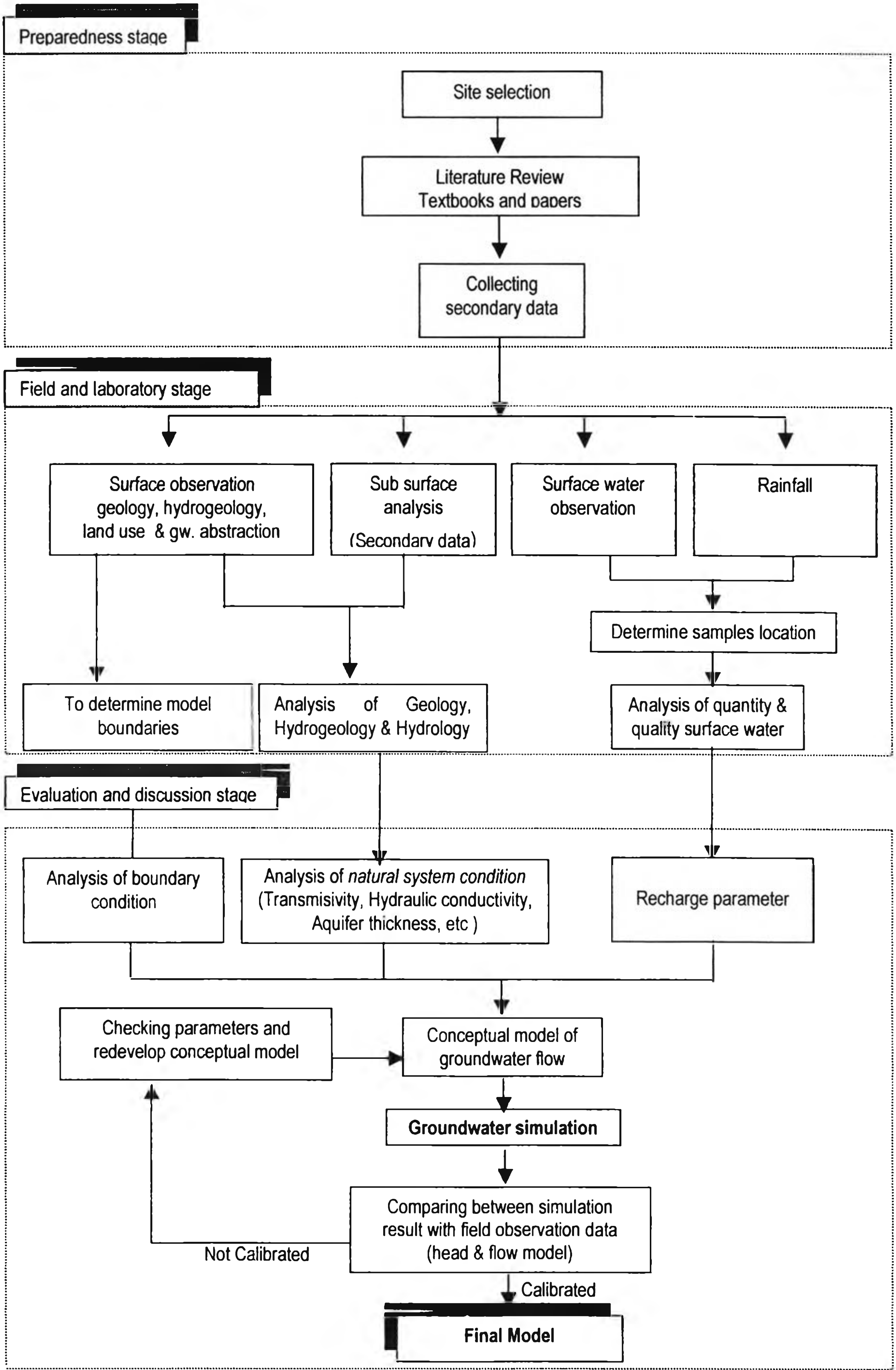

Figure 3.5. Flow chart of research methodology.

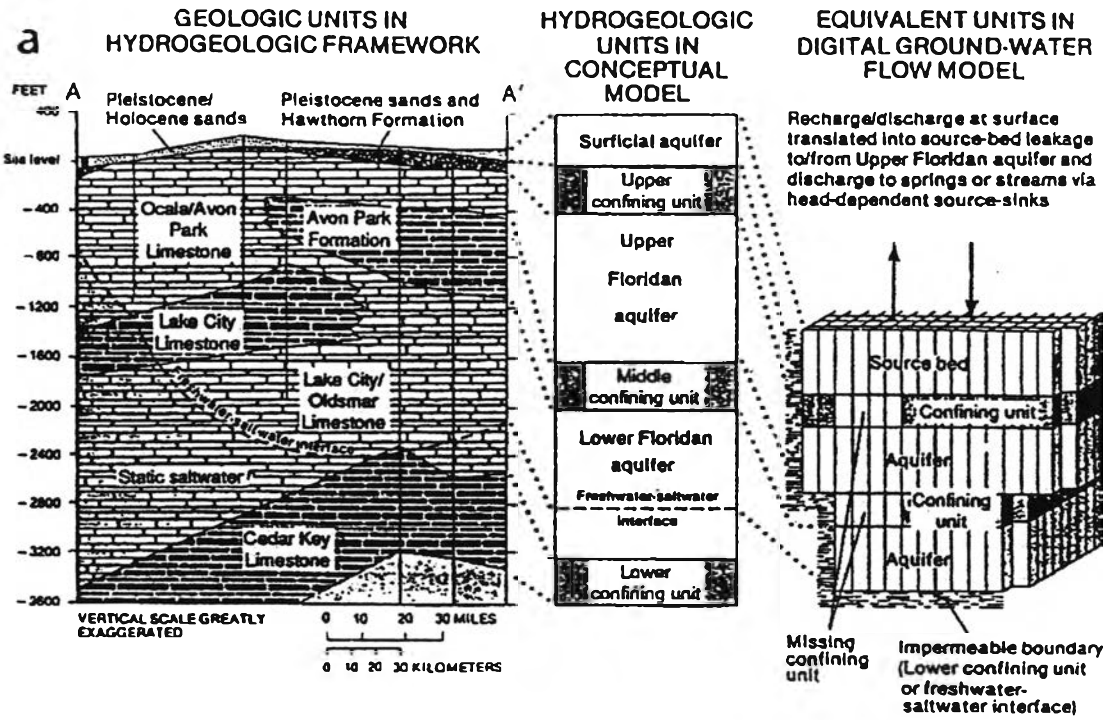

Figure 3.6. Translation of geologic information into a conceptual model suitable for numerical modeling (from Anderson & Woessner, 1992).

one or more of the following steps depending on the nature of the problem being simulated: (1) define hydrogeologic features of interest (e.g. the aquifers to be modeled), (2) define the flow system and sources and sinks of water in the system, (3) define the transport system and source and sinks of chemicals in the system. Figure 3.6. illustrates translation from geologic information into conceptual model.

3.6.2. Discretization

In numerical models, the physical layout of the area in question is replaced with a discretized model domain referred to as a grid and consisting of cells, blocks, or elements depending on whether finite difference of finite element method are used.

Generally, the grid should be drawn on an overlay of a map of the area to be modeled. It is preferable to align the horizontal plane of the grid such that the x and y axes are collinear with K_x and K_y, respectively. The vertical axis of the model, when present, should be aligned with k_z (Anderson and Woessner, 1992). Selecting the size of the cells/elements to be used is a critical step in grid design that depends on many factors such as: spatial variability in model parameters, physical boundaries of system, type of model being used (finite difference or finite element), computer model limitations, data handling limitations, runtime, and associated computer cost. Spatial discretization may affect model results.

Discertization decisions also need to be made for the time parameter. The majority of numerical models calculate result at time t by subdividing the total time into time steps Δt. Generally, smaller time steps are preferable, but the computational time and cost involved in the modeling process increases as the time step is decreased. Time steps may be influenced by the requirements of the model. Some models suffer from numerical instabilities, which cause unrealistic oscillating solutions if a sufficiently small time step is not used. It is a good modeling practice to test the sensitivity of the model results to the size of the time step.

3.6.3. Dimensionality

A good rule of thumb to use when deciding the dimensionality of the model is to avoid complexity if at all possible. For example, air pollution modeling might require a three-dimensional analysis of pollutant dispersion and diffusion. Groundwater contamination at a field site where the data has been collected using conventional monitoring wells, however, it can be simulated only as a 2-D problem because there is not a 3-D definition of the plume of contamination.

3.6.4. Boundary and initial condition

The governing equation alone is not sufficient to describe a specific physical system. This is because a general solution of an n^{th} order differential equation will involve n independent arbitrary constants of functions. In order to define uniquely a given physical problem, the values of the constants or forms of the functions must be specified. Initial and boundary conditions can be used to provide this required additional information. Generally, boundary conditions specify the value of the dependent variable, or the value of the first derivative of the dependent variable, along the boundaries of the system being modeled.

Correct selection of boundary conditions is a critical step in model design. In steady-state simulations, for example, the boundaries largely determine the flow pattern. Boundary conditions affect transient solutions when the effects of the transient stress reach the boundary. In this case, the boundaries must be selected such that the simulated effect is realistic.

Boundary conditions are typically derived from physical and/or hydraulic boundaries of ground water flow systems, for example, the presence of an impermeable body of rock or a river in connection with ground water aquifer. Three types of mathematical formulation represent hydrogeologic boundaries: specified head, specified flux, and head dependent flux boundaries.

Specified head boundary (Dirichlet conditions); A specified head boundary is simulated by setting the head at the relevant location equal to known values:

$$H(x,y,z) = H_0 \tag{3.28}$$

Where $H(x,y,z)$ is the head at a point with coordinates (x,y,z) and H_0 is a specified head value. It is important to recognize that a specified head boundary represents an inexhaustible supply of water.

Specified flux boundary (Neumann conditions); Specified flux boundary is defined by giving the derivative of the head across the boundary:

$$q_x = \frac{\delta H}{\delta x} = Cons\tan t \tag{3.29}$$

This type of boundary is used to describe fluxes to surface water bodies, spring flow and seepage to and from bedrock underlying the system. A special type of specified flux boundary is a no-flow boundary, which is set specifying flux to be zero. A no-flow boundary may represent impermeable bedrock, an impermeable fault zone, a ground water divide or a streamline.

Head-dependent flux boundary (Cauchy or mixed conditions). For this type of boundary, the flux across the boundary is calculated given a boundary head value:

$$\frac{\delta H}{\delta x} + \delta H = C \tag{3.30}$$

where *a* and *C* are constants. Leakage to or from a river, for instance, can be simulated using this type of boundary condition.

In some instances, it may not be possible to use physical boundaries and regional groundwater divides. Other hydraulic boundaries can be defined from information on the configuration of the flow system. However, care must be taken when establishing such boundaries to ensure that the model boundaries will not cause the solution to differ significantly from the response that would occur in the field. For example, hydraulic boundaries may be defined from a water table map of the area to be modeled. The model grid is superimposed on the water table contour map, and specified head boundary conditions can be interpolated. It is important to verify, however, that these boundary conditions will not be impacted by stresses imposed on the model, such as pumping from a location near the boundary.

3.6.5. Source and sinks

Water as well as chemical may enter the grid in one of two ways through the boundaries, as determined by the boundary conditions, or through source and sinks within the interior of the grid. Even though the same model options may be used to represent boundary sources and sinks as to represent internal sources and sinks, the modeler should remember that internal sources and sinks are not boundary conditions. For example, specified head cells are used to represent specified head boundary conditions, but specified head nodes may be placed within the grid to represent lakes and rivers or some other type of source.

An injection or pumping well is a point source or sink and is represented in a ground water model by specifying an injection or pumping rate at a designated node or cell. In a 2-D model, an assumption of a fully penetrating well over the aquifer thickness is made. The prospective modeler is cautioned when modeling well with models that allow only a uniform grid (i.e., all cells have the same size), and when the cell size in the model greatly exceeds the actual diameter of the well. The head calculated by the model is not an accurate approximation of the head measured in the well, but rather the head value predicted by the model is closer to an average to the heads measured as one moves outward from the well toward the edge of the cell.

3.6.6. Model calibrations and sensitivity

Calibration a model is the process of demonstrating that the model is capable of producing field-measured values of the unknown model variable. For the case of groundwater flow, for example, calibration is accomplished by finding a set of parameters, boundary and initial conditions, and stresses that produce simulated values of heads and/or fluxes that match measured values within a specified range of error. There are two ways to achieve model calibration: manual trial-and-error selection of parameters; and automated parameter estimation. In trial-and-error calibration, parameter values are initially assigned to the grid. The initial parameter values are adjusted in sequential model runs to match simulated data to the calibration targets. The trial-and-error process is quite subjective and is influenced by the modeler's expertise. Automated inverse modeling may not be subjective and is not influenced by the modeler; however, it suffers from being complicated and computer intensive. With the indirect approach, an inverse code automatically checks the head solution and adjusts parameters in a systematic way so as to minimize an objective function, which compares the model-caiculated values of head to the measured values.

The results of the calibration should be evaluated relative to the measured values both qualitatively and quantitatively. A quantitative evaluation of the calibration involves listing the measured and simulated values and determining some average of the algebraic differences between them. Two methods are commonly used to express these differences:

$$\text{Mean Error} = \frac{1}{n}\sum_{i=1}^{n}(x_m - x_s)_i \qquad (3.31)$$

$$\text{Root Mean Squared Error} = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(x_m - x_s)_i^2} \qquad (3.32)$$

Where x_m and x_s are the measured and simulated values, respectively.

The purpose of a sensitivity analysis is to quantify the effects of uncertainty in the estimates of model parameters on model results. During a sensitivity analysis, calibrated values for hydraulic conductivity, recharge, boundary conditions, etc., are systematically changed within a pre-established range of applicable values. The results of the sensitivity analysis are expressed as the effects of the parameter change on the average measure of error (mean error or root mean square error) and on the spatial distribution of heads and/or concentrations.

3.6.7. Model verification and prediction

Because of uncertainties in parameter estimates for a given site, the calibrated model parameters may not accurately represent the system under a different set of boundary conditions or hydrologic stresses. In a typical verification exercise, values or parameters and hydrologic stresses determined during calibration are used to simulate a transient response for which a set field data exists. Unfortunately, sometimes it is not possible to verify a model because only one data set exists and it is usually used in the calibration process. A calibrated but unverified model can still be used to make predictions as long as sensitivity analyses of both the calibrated and predictive model are performed and evaluated.

Prediction is one of the more common motivations for modeling. In a predictive simulation, the parameters determined during calibration are used to predict future conditions or the response of the system to future events. The length of time for which prediction may be required is an important consideration in model selection and design. The prediction process should be associated with a sensitivity analysis similar to that completed after calibration.

3.7. Groundwater Balance

Under natural condition, an aquifer is usually in a state of dynamic equilibrium (Theis, 1983). A volume of water recharges the aquifer and an equal volume is discharged. The potentiometric surface is steady and the amount of water in storage in the aquifer is constant.

Calculation of safe yield in an area involves relating geohydrologic and operational factors in a quantitative form, known as a "water balance" or "hydrologic budget". The hydrologic budget may be written as:

$$Inflow = outflow \pm change\ in\ storage \qquad (3.33)$$

Equation 4.32 is commonly known as the "equation of hydrologic equilibrium" and is an example of the law of conservation of matter. Although the concept is invariant, the particular use may vary slightly in form and complexity depending on application. Inflow includes all precipitation infiltration, and other recharge. Outflow includes exploitation, evapotranspiration, and seepage losses to the surface or adjacent ground water reservoirs. The following example employs an equation of the form.

$$Q_i + Q_{cl} + Q_{ur} + EP + Q_{rfs} + Q_{rfg} = Q_o + Q_{ip} \pm \Delta V \qquad (3.34)$$

Where,

Q_i = Ground water inflow $[L^3T^{-1}]$

Q_{cl} = Conveyance loss from unlined surface canals contributing to ground water Recharge $[L^3T^{-1}]$

Q_{ur} = Recharge from unaccounted sources and inter-aquifer leakage $[L^3T^{-1}]$

EP = Effective precipitation recharging the ground water reservoir $[L^3T^{-1}]$

Q_{rfs} = Return flow from surface water irrigation $[L^3T^{-1}]$

Q_{rfg} = Return flow from groundwater irrigation $[L^3T^{-1}]$

Q_o = Groundwater outflow $[L^3T^{-1}]$

Q_{ip} = Groundwater pumping $[L^3T^{-1}]$

ΔV = Groundwater storage change $[L^3T^{-1}]$

The amount of water that recharges an unconfined aquifer is determined by tree factors: the amount of precipitation that is not lost by evapotranspiration and runoff and is thus available for recharge; the vertical hydraulic conductivity of surficial deposits and other strata in the recharge area of the aquifer; and the transmisivity of the aquifer and potentiometric gradient.

Recharge to confined aquifer can occur in places in which the controlling layer is absent. Under such conditions, the three factors affecting unconfined aquifer recharge are controlled. Recharge to a confined aquifer may come from both down flow from a higher aquifer or up flow from a lower aquifer.

When a well begins to pump water from aquifer, the water is withdrawn from storage around the well and from vertical leakage (Theis, 1938). As the cone of depression grows, an increasingly larger portion of the aquifer will be contributing water from storage. In any event, the pumping cone will continue to grow until it has sufficiently reduced natural discharge or increased recharge to balance the volume or water removed by pumping. With this occurrence, a new condition of dynamic equilibrium is reached. The rate at which the cone of depression spreads is a function of the storativity of a confined aquifer or the specific yield of an unconfined aquifer.

3.8. Schedule of Research

Detail of research schedule is listed in Table 3.2.

Table 3.2. Schedule of research.

No.	Task	Time (Month)									
		2003						2004			
		6	7	8	9	10	11	12	1	2	3
1.	Data Collection										
	• Secondary data										
	• Primary data (if necessary)										
2.	Data Analysis										
	• Geological and hydrogeological setting										
	• Aquifer properties										
	• Rainfall										
	• Groundwater abstraction										
	• Groundwater quality										
3.	Groundwater Modeling										
	• Determine model parameter										
	• Conceptual model										
	• Simulating groundwater flow										
	• Model calibration										
	• Water balance										
4.	Final Model										
5.	Interpretation and Recommendation										
6.	Report										