- The concentration of salt in the groundwater;
- The concentration of salt in the soil layers above the watertable (i.e. in the unsaturated zone);
- The spacing and depth of the wells;
- The pumping rate of the wells;
- The percentage of tubewell water removed from the project area via surface drains.

The first two of these factors are determined by the natural conditions and the past use of the project area. The remaining factors are engineering-choice variables (i.e. they can be adjusted to control the salt build-up in the pumped aquifer).

A common practice in this type of study is to assess not only the project area's total water balance (Chapter 16), but also the area's salt balance for different designs of the tubewell system and/or other subsurface drainage systems.

22.4 Basic Equations

Chapter 10 described the flow to single wells pumping extensive aquifers. It was assumed that the aquifer was not replenished by percolating rain or irrigation water. In this section, we assume that the aquifer is replenished at a constant rate, R, expressed as a volume per unit surface per unit of time (m^3/m^2d = m/d). The well-flow equations that will be presented are based on a steady-state situation. The flow is said to be in a steady state as soon as the recharge and the discharge balance each other. In such a situation, beyond a certain distance from the well, there will be no drawdown induced by pumping. This distance is called the radius of influence of the well, r_e.

If more wells are used to drain an area, the pattern and spacing of the individual wells will determine the water level in the well field and the drawdown of the water level in the individual wells. Wells should be placed in such a way that the water level is lowered sufficiently everywhere in the area.

Drainage equations are presented for tubewells placed in two regular patterns:
- A triangular pattern. This is hydraulically the most favourable well-field configuration, with a maximum area to be drained by one well and with no extra drawdown induced by neighbouring wells. The disadvantage of a triangular configuration is that more length of collector drains is required to transport the water to the main collectors (see Section 22.5.2);
- A rectangular pattern, in which the wells are placed along parallel collector drains. For this well-field configuration, a minimum length of collector drains is required (see Section 22.5.2). The disadvantage of a rectangular configuration is that interference from neighbouring wells will cause extra drawdown to occur in the wells, leading to somewhat higher pumping costs.

22.4.1 Well Field in a Triangular Pattern

When the wells in a well field are placed in a triangular pattern, their individual radii of influence hardly overlap, as can be seen from Figure 22.3. The simplifying

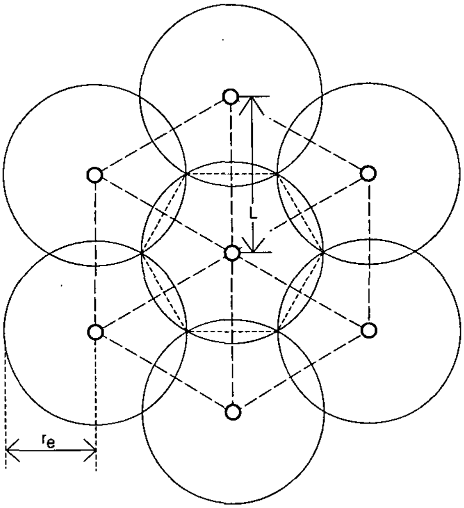

Figure 22.3 Wells located in a pattern of equilateral triangles (well spacing $L = r_e\sqrt{3}$)

assumption is then made that the discharge and the drawdown of each well will not be affected by those of neighbouring wells. In other words, the theory of a single well can be used.

In a drainage well field, there is a direct relationship between the discharge rate of the well, the recharge rate of the aquifer by percolation, and the area affected by pumping. The decline of the water level due to pumping is determined by the discharge rate of the well and the permeability and thickness of the aquifer. The discharge rate and the drawdown in the well are important factors in calculating the pumping costs of well drainage.

In an unconfined aquifer, the steady-state flow through an arbitrary cylinder at a distance r from the well is given by

$$Q_r = \pi\,(r_e^2 - r^2)\,R \qquad (22.2)$$

where

r_e = radius of influence of the well (m)
R = recharge rate of the aquifer per unit surface area (m/d)

According to Darcy's law (Chapter 7), Q_r equals the algebraic product of the cylindrical area of flow and the flow velocity. Hence, the discharge at distance r from the well can also be expressed by

$$Q_r = 2\,\pi\,r\,h\,K\,\frac{\delta h}{\delta r} \qquad (22.3)$$

where

K = hydraulic conductivity of the aquifer (m/d)
$\delta h/\delta r$ = hydraulic gradient in the aquifer at distance r (–)

Since, in steady state, the discharge of the well, Q, equals the vertical recharge of the area within the radius of influence, the following relationship can be used

$$Q = \pi r_e^2 R \tag{22.4}$$

Combining Equations 22.2 and 22.4 yields

$$Q_r = Q - \pi r^2 R \tag{22.5}$$

or, combining Equations 22.3 and 22.5 and separating r and h

$$\left(\frac{Q}{r} - \pi r R\right) \delta r = 2 \pi K h \delta h \tag{22.6}$$

Integration between the limits $r = r_w$, $h = h_w$, and $r = r_e$, $h = h_e$ yields

$$Q \ln\left(\frac{r_e}{r_w}\right) - \frac{1}{2} \pi R (r_e^2 - r_w^2) = \pi K (h_e^2 - h_w^2) \tag{22.7}$$

The quantity $1/2\ \pi R r_w^2$ is very small in comparison with $1/2\ \pi R r_e^2$ and can be neglected. If, moreover, the drawdown in the well is small in comparison with the original hydraulic head, the right-hand side of Equation 22.7 can be expressed as (Peterson et al. 1952)

$$\pi K (h_e + h_w)(h_e - h_w) \approx \pi K 2 H (h_e - h_w) = 2 \pi K H \Delta h_r \tag{22.8}$$

where

H = saturated thickness of the aquifer before pumping (m)
Δh_r = drawdown due to radial flow towards the pumped well (m)

Since, according to Equation 22.4,

$$r_e^2 = \frac{Q}{\pi R} \tag{22.9}$$

Equation 22.7 can be written as

$$\Delta h_r = \frac{Q}{2 \pi K H}\left[2.3 \log\left(\frac{r_e}{r_w}\right) - \frac{1}{2}\right] \tag{22.10}$$

If $r_e / r_w > 100$, and if we accept an error of 10%, the term –1/2 can be neglected and Equation 22.10 reduces to

$$\Delta h_r = \frac{2.3\, Q}{2 \pi K H} \log \frac{r_e}{r_w} \tag{22.11}$$

Equation 22.11 can be used to calculate the drawdown in a well field when the wells are placed in a triangular pattern. From Figure 22.3, it can be seen that the distance L between the wells is then equal to $r_e \sqrt{3}$.

Example 22.1
In an irrigated area, it has been estimated that the average deep percolation losses resulting from excess irrigation water amount to 2 mm per day.

The hydraulic conductivity of the aquifer is K = 25 m/d; the thickness of the water-bearing layer is H = 25 m. The radius of each well is $r_w = 0.1$ m.

Suppose the wells are to be placed in a triangular pattern, 1000 m apart. What will be the required pumping rate of each well and what will be the drawdown in each well?

According to Figure 22.3, the radius of influence will be

$$r_e = 1000 / 1.73 = 578 \text{ m}$$

The discharge rate of each well is given by Equation 22.4

$$Q = 3.14 \times 578^2 \times 0.002 = 2098 \text{ m}^3/\text{d}$$

Substituting this value into Equation 22.11 gives

$$\Delta h_r = \frac{2.3 \times 2098}{2 \times 3.14 \times 25 \times 25} \log \frac{578}{0.1} = 4.6 \text{ m}$$

The drawdown in each well is thus 4.6 m.

22.4.2 Well Field in a Rectangular Pattern

The formulas discussed so far apply only to wells forming triangular patterns. They are not applicable to wells sited in parallel lines at a distance B apart. The spacing of the wells along the lines is L, where L is considerably smaller than B (Figure 22.4).

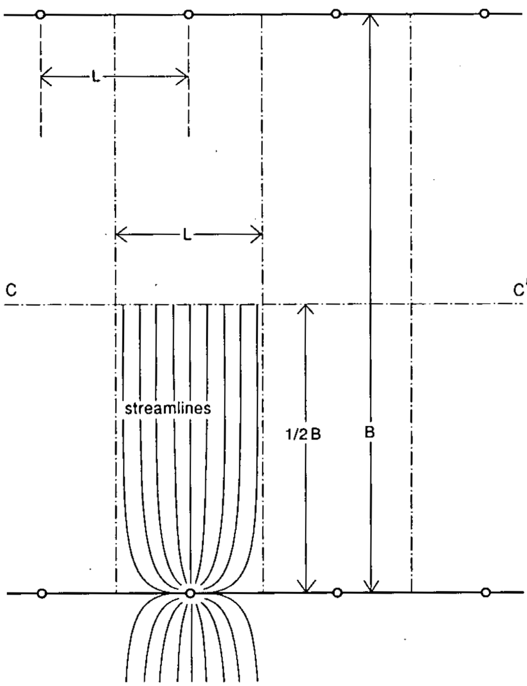

Figure 22.4 Wells in parallel series a distance B apart. Well spacing within the series is L with L< <B (after Edelman 1972)

In such a situation, if the recharge on the land surface from rain or irrigation water is uniform, and if the flow towards the wells has attained a steady state, the discharge of each well can be written

$$Q = R\,B\,L \tag{22.12}$$

where Q is the discharge rate of each well in m^3/d.

As parallel lines of wells show a certain analogy with parallel ditches or canals, Edelman (1972) derived an approximate solution for the drawdown at the face of each well. In both cases, the watertable is lowered along a line, which is the axis of either the line of wells or the ditch or canal. Hence the lines of wells can be replaced by ditches or canals from which a quantity q_o (m^2/d) is extracted per unit length, so that

$$q_o = R\,B \tag{22.13}$$

The maximum watertable height occurs in the symmetry axis, $C-C'$. The difference in hydraulic head (i.e. the difference between the maximum watertable elevation midway between the ditches or canals and the water level in them, also called available head) is given by (analogous to Equation 8.6 in Chapter 8)

$$\Delta h_h = \frac{R\,B^2}{8\,K\,H} \tag{22.14}$$

In reality, of course, the extraction does not take place from canals or ditches, but from parallel lines of wells. As a consequence, the hydraulic head midway between the lines of wells (in the symmetry line $C-C'$) is not constant. Deviations from the average value of the head can be neglected, however, because it was assumed that the distance B between the lines is much greater than the well spacing L along the lines. As can be seen in Figure 22.4, the streamlines cross the line of symmetry, C – C', almost at right angles. Hence the head midway between the lines of wells can be considered a constant, h_e. In addition, the hydraulic head in a well, h_w, is lower than the head in the canal. The energy losses are concentrated in the vicinity of the well, where the flow is radial.

For radial flow, the drawdown can be expressed as

$$\Delta h_r = \frac{2.3\,Q}{2\,\pi\,K\,H} \log\frac{r_e}{r_w} \tag{22.15}$$

The method of superposition can be applied to find the difference between the head at the well face and that midway between the lines of wells. Combining Equations 22.14 and 22.15 gives

$$\Delta h = \frac{R\,B^2}{8\,K\,H} + \frac{2.3\,Q}{2\,\pi\,K\,H} \log\frac{r_e}{r_w} \tag{22.16}$$

Taking for r_e such a value that the circumference of a circle with radius r_e is equal to the length of the section through which the water flows from both sides towards the well

$$2\,\pi\,r_e = 2\,L$$

we can rewrite Equation 22.16 as

$$\Delta h = \frac{R\,B^2}{8\,K\,H} + \frac{2.3\,Q}{2\,\pi\,K\,H} \log \frac{L}{\pi\,r_w} \tag{22.17}$$

Equation 22.17 can be used to calculate the head loss in a well field when the wells form a rectangular pattern. Such a pattern is recommended when surface drains in parallel lines already exist in the drainage area.

Example 22.2
Suppose that, in the same area as described in Example 22.1, the surface drains are situated 2000 m apart. Assuming the same pumping rate, what will be the distance between the wells and what will be the drawdown in each well?

According to Equation 22.12, the distance between the wells is

$$L = \frac{2098}{0.002 \times 2000} = 525\,\text{m}$$

Substituting the appropriate values into Equation 22.17 gives

$$\Delta h = \frac{0.002 \times 2000^2}{8 \times 25 \times 25} + \frac{2.3 \times 2098}{2 \times 3.14 \times 25 \times 25} \log \frac{525}{3.14 \times 0.1} =$$

$$= 1.6 + 4.0 = 5.6\,\text{m}$$

The drawdown in each well is thus 5.6 m.

22.4.3 Partial Penetration

The equations presented in the previous two sections were derived under the assumption that the wells fully penetrate the pumped aquifer. Some aquifers are so thick, however, that installing a fully-penetrating well would not be justified. In these cases, the aquifer has to be pumped by a series of partially-penetrating wells.

Partial penetration causes the flow velocity in the immediate vicinity of the well to be higher than it would otherwise be, leading to an extra loss of head. According to Hantush (1964), the effect of partial penetration in an unconfined aquifer is similar to that in a confined aquifer (Chapter 2), provided the drawdown is small in relation to the saturated thickness of the aquifer, H.

Under these conditions, the formula developed for confined aquifers can also be applied to unconfined aquifers, provided the calculated additional head loss is corrected according to Jacob (see Chapter 10, Equation 10.6). In many cases, this correction would result in differences of some millimetres only and can therefore be ignored.

In view of the above, the formula for calculating the effect of partial penetration (Hantush 1964) reads

$$\Delta h_p = \frac{Q}{4\,\pi\,K\,H}\,F \tag{22.18}$$

in which

$$F = 2\frac{H}{p}\left[\left(1 - \frac{p}{H}\right)\ln\left(\frac{2\,p}{r_w}\sqrt{\frac{K_h}{K_v}}\right) - \frac{p}{H}\ln\frac{2\,H}{p} - 0.423\frac{p}{H} + \ln\frac{2\,H + p}{2\,H - p}\right] \qquad (22.19)$$

where

p = penetration depth of the well into the aquifer (m), assuming screening over the full depth of the well
K_h = horizontal hydraulic conductivity (m/d)
K_v = vertical hydraulic conductivity (m/d)

and the other symbols as previously defined.

So, when the wells in the proposed well field only partially penetrate the aquifer, the additional head loss calculated from Equation 22.18 should be added to the drawdowns calculated by Equations 22.11 or 22.17, depending on the actual configuration of the well field.

Example 22.3
The additional head loss due to partial penetration will only have a substantial value if the penetration ratio is relatively low. The penetration ratio is defined as the ratio of penetration depth of the well into the aquifer and the thickness of the aquifer. For that reason, the aquifer thickness is not taken as 25 m as in Example 22.1, but as 300 m.

Assuming that the wells are still 25 m deep and that the K_h/K_v ratio is 25, what will be the additional head loss due to partial penetration?

According to Equation 22.19, the factor F will be

$$F = \frac{2 \times 300}{25}\left[\left(1 - \frac{25}{300}\right)\ln\left(\frac{2 \times 25}{0.1}\sqrt{\frac{25}{1}}\right) - \frac{25}{300}\ln\frac{2 \times 300}{25} - 0.423\frac{25}{300} + \ln\frac{2 \times 300 + 25}{2 \times 300 - 25}\right] = 24 \times 6.955 = 167$$

Substituting this factor into Equation 22.18, together with the other values, then yields

$$\Delta h_p = \frac{2098}{4 \times 3.14 \times 25 \times 300} \times 167 = 3.72\ \text{m}$$

Using Equation 22.11 with H = 300 m and the other parameters as given in Example 22.1, we obtain

$$\Delta h_r = \frac{2.3 \times 2098}{2 \times 3.14 \times 25 \times 300}\log\frac{578}{0.1} = 0.39\ \text{m}$$

The actual drawdown is thus 0.39 + 3.72 = 4.1 m.

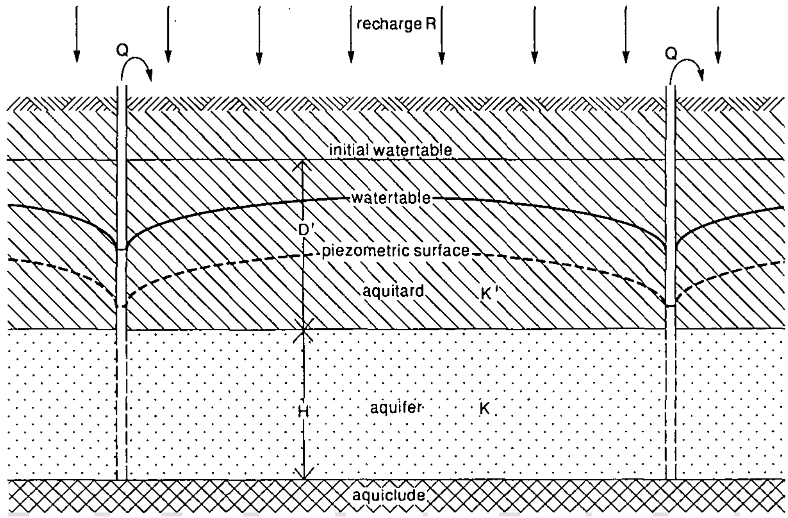

Figure 22.5 Wells in a semi-confined aquifer

22.4.4 Semi-Confined Aquifers

Figure 22.5 shows a semi-confined aquifer whose overlying layer, the aquitard, is replenished by percolating rain or excess irrigation water at a rate R. Depending on the recharge rate and the hydraulic resistance of the aquitard, a difference in head between the free watertable in the aquitard and the piezometric level of the aquifer will develop, as was described by Equation 22.1.

Under steady-state conditions, the same recharge rate will replenish the underlying aquifer. So, Equations 22.11 and 22.17 can also be used to calculate the head loss in a well field when the pumped aquifer is semi-confined.

It should be noted that, with semi-confined aquifers, Equations 22.11 and 22.17 describe the drawdown in the well with respect to the piezometric level of the aquifer. In the calculation of the total water-level depth inside the well (see Section 22.5.2), the difference between this piezometric level of the aquifer and the free watertable in the aquitard should also be considered.

22.5 Design Procedure

The design of a tubewell drainage system depends on a number of physical, technical, practical, and economic parameters. In the design procedure, the following elements can be distinguished: design considerations, well-field design, well design, and design optimization. These elements are described in the following sub-sections.

22.5.1 Design Considerations

Important design considerations are the design discharge of the tubewells, the tubewell operating factor, the annual drainable surplus, and the peak drainage requirement. When applicable, a distinction will be made between autonomous and design factors.

Tubewell Design Discharge
The design discharge rate depends on the autonomous and design factors summarized below.

Autonomous factors are:
- The design should be based on the most economic pump capacity. If larger pumps are installed, fewer pumps will be required, which generally results in lower investment costs. On the other hand, larger capacity pumps result in higher drawdowns and thus higher energy costs. Determining pump capacities on a purely economic basis could lead to very high pumping rates. There are, however, several practical constraints to these high pump capacities;
- The selection of pumps and engines should be based on their availability on the local market; spare parts, especially, should be locally available;
- A policy of reducing the number of different pump sizes may be another major constraint on the choice of the pump capacity;
- A well with a very high pump capacity may serve a very large area that exceeds the spacing determined by other factors. If such a well were to be out of order for a prolonged period, the neighbouring wells would be overburdened, and proper drainage of the area would be impossible;
- If the water is also used for irrigation, pump capacities are often limited by the requirements of the farmers.

Design factors are:
- The annual drainable surplus and the peak requirements. The maximum tubewell capacity will influence the distance between the wells or the maximum spacing in the well field. Hence, for a given operating factor, the drainable surplus would be the determining factor for the discharge rate of the well;
- The horizontal and vertical hydraulic conductivity and the thickness of the aquifer, and the vertical resistance of the aquitard, determine the drawdown for a given discharge rate and the expansion of the cone of depression;
- Screen and casing specifications, together with the discharge rate, determine the entrance velocity of water flowing through the screen, which has a maximum value in order to ensure a maximum lifetime for the well.

Tubewell Operating Factor
The tubewell operating factor is the number of actual operating hours of the well per 24 hours, expressed as a fraction. The tubewell operating factor largely depends on autonomous factors, but also on a design factor like the peak drainage requirement, which will be described below. It will not be possible to operate all wells continuously over an extended period. Time will be lost during maintenance, inspection, and repairs, stoppage due to power failures, etc. Social factors like the presence or absence of a pump operator will also influence the possible operating factor of the wells.

Annual Drainable Surplus

The annual drainable surplus of an area is the annual discharge, in mm/day, required to maintain the design water-level criteria. It is an important design factor in well drainage. It depends on many factors, which are described elsewhere in this publication. Not all these factors apply to tubewell drainage. One such factor is the depth at which the watertable is to be controlled. This design watertable depth depends on:

- The quality of the groundwater;
- The capillary-rise potential of the soil;
- The type of cultivation;
- The type of drainage system.

From a practical point of view, too, the type of drainage system will affect the groundwater-depth criterion. In a pipe drainage project, for instance, a deeper future watertable will have a significantly greater impact on project costs than tubewells will have. Consequently, the drainage criteria and drainable surplus of a pipe drainage project are mainly based on the requirements for cultivated land. For cultivated land and bare soils, watertable depths of 1.0 to 1.5 m below the surface are widely applied, although a deeper watertable might be preferable in view of a better control of salinity and waterlogging. In fallow land, any extra capillary rise of salt is counteracted by applying extra irrigation water for leaching before the start of the seasonal cultivation of the land.

Under tubewell drainage, the requirements for the depth of the watertable are more demanding for fallow lands than for cultivated lands. When fields are cultivated, unavoidable field losses percolating through the soil profile wash the salts downward, whereas, in fallow lands, the capillary rise causes the salts to move upward, which may impair the cultivation of the next crop. As the area drained by a tubewell is relatively large (up to some 500 ha), each tubewell nearly always serves fallow as well as cultivated land. The watertable-depth criterion for drainage by tubewells should therefore be based on the requirement for fallow land.

To avoid re-salinization of the soil under fallow conditions, the groundwater should be drained to a deeper level, say 1.8 m to 2 m, depending on the type of soil (see Chapter 11), thereby increasing the drainage requirements. In principle, the same drainage depth should also be taken for pipe drainage, but the extra investment costs would make this prohibitive, so a shallower depth is applied. The deeper level is also required to offset any irregularities in the topography of the area served by the tubewell. This means that, in the field, the deeper level may be exceeded in some areas, while in others it will never be reached.

Peak Drainage Requirement

The recharge to the aquifer in an irrigation area will vary throughout the year, depending on the water supplies to the area. For an area with an annual average recharge of 1 mm/d, the minimum and maximum recharges may, for example, be 0.5 and 1.5 mm/d respectively. Other seasonal fluctuations may be due to different irrigation requirements for perennial and seasonal crops. In areas with tubewell drainage, the resulting differences in recharge cause the actual watertable depth to vary through the year. In areas where groundwater is pumped purely for drainage purposes, the seasonal fluctuations may be of the order of 0.5 m.

The peak drainage requirement is the maximum discharge, in mm/d, required for a specified drainage area. The quantification of this design factor was discussed in Chapter 17. The peak drainage rate in an area under pipe drainage may be considerably higher than the mean daily drainage rate because the diameter of drain pipes may allow a peak flow that exceeds, by several times, the annual average flow.

To maintain a stabilized watertable in tubewell drainage, the system ought to be based on the maximum expected recharge. This, however, would result in excessive investment costs. If the system were to be based on a continuous discharge to drain the annual drainable surplus at a constant rate, the watertable would fluctuate throughout the year. This variation can be reduced by adjusting the monthly tubewell operating factor (see Equation 22.20). This means higher operating factors during the periods with higher recharges and lower operating factors during the periods with lower recharges.

In areas with fresh groundwater, where the pumped water is also used for irrigation, seasonal fluctuations may be much greater, namely of the order of 1 to 3 m. If the seasonal water-level fluctuations are no impediment to agriculture and the aquifer is large enough to store the peaks in recharge by infiltrating irrigation water, the peak drainage requirements can be excluded from the design considerations.

22.5.2 Well-Field Design

The distance between the wells in the different well-field configurations can be calculated on the basis of the factors discussed in the previous section.

Well-Distance Calculation Procedure

In a tubewell field, the spacing between the wells and the well-field configuration depend on various differing design considerations, which will be discussed below.

The operating factor and the discharge rate determine how much water will be pumped by one tubewell. In combination with the drainable surplus, they determine the drainage area per tubewell and thus also the number of tubewells required for the total drainage area. This can be expressed in the following equation

$$A_w = \frac{0.1\ Q\ t_w}{q} \tag{22.20}$$

where

A_w = drainage area per well (ha)
Q = discharge rate of the well (m^3/d)
q = drainable surplus (mm/d)
t_w = tubewell operating factor (−)

The total number of wells required can be found by dividing the total drainage area by the drainage area per tubewell. Equation 22.20 shows that the discharge rate of a well is directly related to the area that can be drained by one well, and thus determines the total number of wells required. Peak drainage requirements occurring over shorter periods can be met by temporarily longer pumping and thus a temporarily higher tubewell operating factor.

Well-Field Configuration

Section 22.4 presented equations for different well-field configurations. For a triangular well-field configuration, the distance between the wells for a selected discharge rate can be calculated by

$$L = 100 \sqrt{\frac{3 A_w}{\pi}} \tag{22.21}$$

and for a rectangular well-field configuration with wells placed along the parallel main drains by

$$L = 10\,000 \frac{A_w}{B} \tag{22.22}$$

where

L = the distance between the wells (m)

B = the distance between the lines of wells (m). (B represents the main drains and is specified by the engineer.)

From a drainage point of view, the ideal well-field layout is the triangular grid system shown in Figure 22.6. In this system, the area of influence of a single well is a hexagon, roughly resembling a circle of the same area with an effective radius, r_e, equal to $1/\sqrt{3}$ times the distance between the wells. This configuration, however, has the following disadvantages:

- The tubewells have to be connected to the main drainage system by means of field drains, as shown in Figure 22.6. For a triangular grid, the required total length of these drains considerably exceeds that for wells placed in a rectangular grid along open main drains as shown in Figure 22.7;
- The cost of electrifying a triangular grid of tubewells is higher than for a line layout parallel to the drains;
- Additional tracks have to be constructed to the tubewells to permit the easy access needed for proper tubewell maintenance.

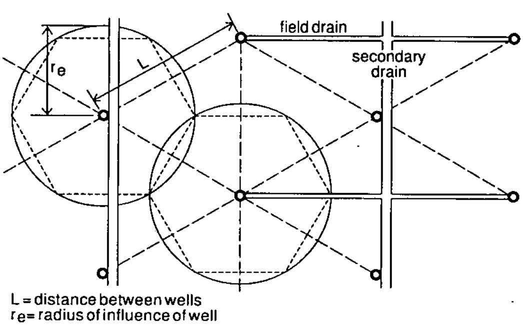

Figure 22.6 Wells in a triangular configuration with the required lay-out of field drains and main drains for the discharge of drainage water

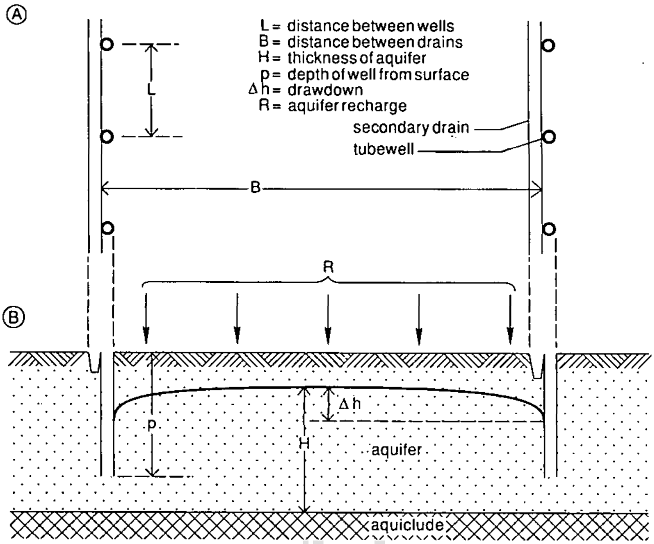

Figure 22.7 Wells in a rectangular configuration with the required lay-out of main drains for the discharge of drainage water
A: plan view
B: cross-section

Example 22.4
An irrigated area of 2500 ha has an annual drainage requirement of 480 mm. The drainable surplus is thus 1.5 mm/d. The maximum running hours of the pump per day are taken to be 15 hours, thus the tubewell operating factor, t_w, equals 0.63.

Suppose that, given the availability of pumps and spare parts, and a policy of reducing the number of different pump sizes, it has been decided to use three different pump capacities: 100, 200, and 300 m^3/h.

According to Equation 22.20, the area drained per well for a discharge rate of 200 m^3/hr is then

$$A_w = \frac{0.1 \times 200 \times 24 \times 0.63}{1.5} = 200 \text{ ha}$$

Substituting this value of A_w into Equation 22.21 gives the spacing of tubewells in a triangular well-field configuration

$$L = 100 \sqrt{\frac{3 \times 200}{3.14}} = 1382 \text{ m}$$

Substituting the value of A_w into Equation 22.22 gives the spacing of tubewells in a rectangular well-field configuration. (Assume the spacing between the main drains to be 5000 m.)

$$L = 10\,000 \frac{200}{5000} = 400 \text{ m}$$

Table 22.2 Well spacings for different pump capacities and well-field configurations

Pump capacities	Area per well	Well spacing	
		Triangular	Rectangular
(m^3/h)	(ha)	(m)	(m)
100	100	977	200 × 5000
200	200	1382	400 × 5000
300	300	1693	600 × 5000

Table 22.2 lists the drainage area per well and the distances between the wells for both well-field configurations and for the above-mentioned three pump capacities.

22.5.3 Well Design

Knowing the discharge rate of the well and having data on lithology and aquifer characteristics (plus the dimensions and properties of available screens and casings), we can design a well.

The principle objectives of a properly designed tubewell are:
- Pumping of water at the lowest cost;
- Pumping of water that is free of sand;
- Minimum operation and maintenance costs;
- A long and economic lifetime.

A good well design depends on many factors, some of which are discussed below. More detailed information on technical well design and construction methods can be found in reference books such as those of Driscoll (1986) and Huisman (1975). Figure 22.8 shows a typical tubewell design.

Considerations on Well Depth

The total depth of a tubewell is determined by the lengths of the pump housing, production casing, screen section, and sand trap (Figure 22.8). The following points should be considered:
- The length of the pump housing should be chosen so that the pump remains below the water level in the well, for the selected discharge rate, under all conditions, and over the total lifetime of the well;
- The length of the production casing (i.e. the section of blind pipe between the bottom of the pump housing and the top of the aquifer) depends on the actual thickness of the aquitard overlying the aquifer. The production casing is not required in unconfined aquifers at shallow depth where the pump housing penetrates deep enough into the top section of the aquifer;
- The length of the screen section depends on the required total screen length and the total length of sections of blind pipe to case off unproductive layers in the aquifer;
- The length of the sand trap (i.e. the section of blind pipe at the bottom of the screen section) is usually of the order of a few metres.

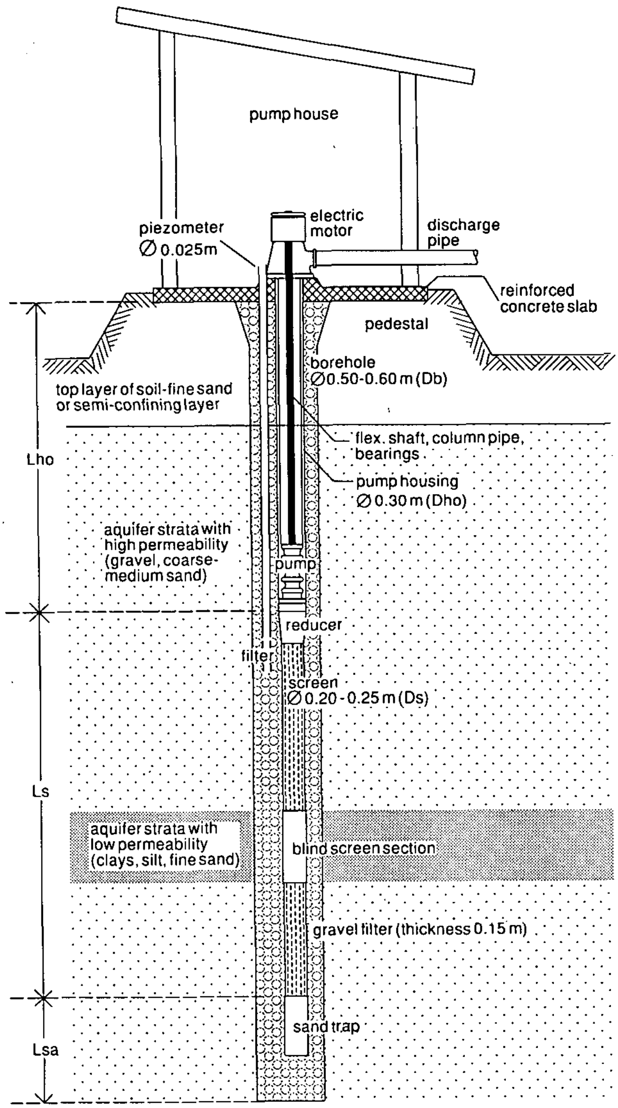

Figure 22.8 Typical design of a tubewell

Considerations on Well Diameter

The diameter of the well depends on the following:

- In the upper section, on the diameter of the pump housing and some angular space, say 25 mm, all around the casing;
- Below the pump-housing, on the diameter of the production casing, if required;
- In the screen section, on the diameter of the screen. Twice the thickness of the gravel pack should be added to this value. For reasons of construction, the minimum thickness of the gravel pack should be 75 mm;
- The diameter of the sand trap is usually the same as that of the screen section.

The purpose and design of these well sections, and their position in the well, will be discussed below.

Pump Housing

The pump housing is the upper section of blind casing that supports the well against collapse, and in which the pump is installed. The length and diameter of the pump housing should be such that it can accommodate the pump at the required depth throughout the lifetime of the well. A pump housing is always required when submersible pumps are used. No special pump housing is required in the case of a shallow watertable with little drawdown where suction pumps can be used; both pump and engine are installed at the surface beside the tubewell.

The actual length of the pump housing is primarily determined by the required depth of the pump. The location of the pump depends on the expected depth to which the water level inside the well will drop for the selected design discharge rate. This water-level depth inside the pumped well depends on the following factors:

- The required design depth to the watertable for the selected discharge rate (see Chapter 17);
- The difference in head between the watertable in the overlying aquitard and the piezometric head in the pumped aquifer when the aquifer is semi-confined;
- The formation losses;
- The well losses;
- The seasonal fluctuations of the watertable, especially when the groundwater is used for irrigation;
- A safety margin.

The difference in head between the free watertable in the overlying aquitard and the piezometric head of the pumped aquifer is determined by the hydraulic resistance of the aquitard. Depending on the actual recharge rate (= drainable surplus), Equation 22.1 can be used to estimate this head loss. It will be clear that this component is not present in unconfined aquifers.

The formation losses are the head losses due to the laminar flow of water to the well, and are determined by the hydraulic conductivity and thickness of the aquifer. Depending on the actual well-field configuration, Equations 22.11 and 22.17 can be used to estimate these losses. With partially-penetrating tubewells, the additional head loss according to Equation 22.18 should be added to the formation losses.

The well losses are the head losses due to turbulent flow in and around the pumped well, and due to a reduced hydraulic conductivity of the aquifer in a zone immediately surrounding the well. This zone is called 'skin' or 'zone of damage'. It is created by the invasion of drilling fluids, the dispersion of clays, the presence of a mud cake, partial penetration, or the clogging of screens. The total well losses consist of head losses in the gravel pack, head losses due to the partial perforation of the screen, and friction losses inside the well. Their calculation is rather complicated; Huisman (1975) presented methods to estimate these expected well losses.

It should be noted that, in principle, the well losses can also be determined from a special type of well test, 'the step-drawdown test', which has been described by Kruseman and De Ridder (1990), among others. This method cannot be used here, however, because the wells to be designed for the well field will usually be different in design from the already existing ones for which the step-drawdown tests were made.

The depth of the pump is determined by the maximum depth of the water level during pumping over the total lifetime of the well, plus the length of the pump and the engine, plus a safety margin of several metres.

The diameter of the pump housing should be large enough to accommodate the pump with clearance of approximately 25 mm all around the pump for its installation and efficient operation. The diameter of the pump depends on the selected discharge rate and the pump type.

Production Casing

The production casing is the section of blind pipe between the bottom of the pump housing and the top of the aquifer. The production casing is not required in unconfined aquifers at shallow depth where the pump housing reaches sufficiently deep into the top section of the aquifer, which is usually the case in drainage projects. The length of the production casing depends on the thickness of the aquitard overlying the pumped aquifer. The diameter of the production casing:
- Is smaller than the diameter of the pump housing;
- Is larger or equal to the diameter of the underlying screen section;
- Must be sufficient to ensure that the upward velocity of pumped water in the casing is less than 1.5 m/s.

Screen Section

A screen has to perform the following functions in a well. It should:
- Support the wall of the well against collapse;
- Prevent sand and fine material from entering the well during pumping;
- Secure a low head loss of water flowing through the slot openings and through the screen;
- Provide resistance to chemical and physical corrosion by the pumped water.

To achieve the above, the screen should have the following properties:
- A large percentage of open area to minimize the head loss and entrance velocity;
- Sufficient column strength to prevent collapse;
- Non-clogging slots;
- Be resistant to corrosion;
- A minimum encrusting tendency.

It is not always possible to combine all these properties. For example:
- An increase in the open area of a screen weakens its column strength;
- PVC and fibreglass screens are lighter and more resistant to corrosion by chemically aggressive water, but have a lower collapse strength than steel screens and casings. In practice, PVC and fibreglass-reinforced screens and casings will be technically and economically attractive in drainage wells in alluvial aquifers, where wells are placed at moderate depths of up to 400 m. Steel screens are required in deep wells drilled in hardrock aquifers. Stainless steel screens combine both strength and resistance to corrosion and chemically aggressive water, but are more expensive;
- The open area of conventional slotted screens should not exceed approximately

10% so as not to weaken the column strength. More expensive continuous slot screens of stainless steel or modern PVC screens have an open area of 30 to 50%, thereby reducing the length of screen required for the minimum entrance velocity. This property is especially important in thin aquifers with a high hydraulic conductivity. If a screen with a relatively low open area is applied in such aquifers, the productivity of the aquifer will ensure a high yield, but the required entrance velocity may limit the maximum allowable pump capacity.

The selection of the screen slot size depends on the type of aquifer and the use of a gravel pack. The screen slot size must be selected so as to ensure that most of the finer materials in the formation around the borehole are transported to the screen and removed from the well by bailing and pumping during the well-development period immediately after the borehole has been constructed and the screen and casing have been installed.

In wells without an artificial gravel pack, well development creates a zone of graded formation materials extending about 0.5 m outward from the screen. Driscoll (1986) and Huisman (1975), among others, give detailed procedures for selecting the correct slot size. They report that with good quality water and the correct slot opening, 60% of the material will pass through the screen and 40% will be retained. With corrosive water the 50%-retained size should be chosen, because even a small enlargement of the slot openings due to corrosion could cause sand to be pumped.

The screen length should be chosen so as to ensure that the actual screen entrance velocity is in accordance with the prescribed entrance velocities as listed in Table 22.3 for the different hydraulic conductivity values of the aquifer.

From these screen entrance velocities, the minimum length of the well screen can be calculated with

$$Q = 86400\, v_e\, l_{min}\, A_0 \tag{22.23}$$

where

Q = discharge rate of the well (m^3/d)
v_e = screen entrance velocity (m/s)
l_{min} = minimum screen length (m)
A_0 = effective open area per metre screen length (m^2/m)

Table 22.3 Recommended screen entrance velocities (U.S. EPA 1976)

Hydraulic conductivity of aquifer (m/d)	Screen entrance velocities (m/s)
> 250	> 0.03
250 - 120	0.03
120 - 100	0.025
100 - 40	0.02
40 - 20	0.015
< 20	< 0.01

In determining the effective open area per metre screen length, it is assumed that 50% of the actual open area is clogged by gravel particles (Huisman 1975). The actual open area per metre screen length depends on the type and diameter of the selected screen type (Example 22.5). The minimum total length of the well screen is one of the most important criteria in well design.

The optimum length of the screen may differ from its minimum length. Determining the optimum screen length is rather complex. It depends on:
- All the cost factors that determine the costs of pumping 1000 m^3 or draining 1 ha;
- The total thickness of the aquifer. In very thick aquifers, the deeper penetration of the well results in a smaller drawdown, which reduces the pumping costs but increases the investment costs in the borehole;
- The selected pumping rate and the system's other design and operating factors.

Finally, the total length of the required screen section is found by adding to the actual screen length, as determined above, the total length of sections of blind pipe used to case off unproductive layers in the aquifer. The total length of blind pipe depends on the distribution of hydraulic conductivity in the aquifer (i.e. the distribution of layers of higher and lower hydraulic conductivity). This stratification can be determined from the driller's log, geophysical logs, and sieve analysis.

The diameter of the screen, like the length and open area of the screen, depends on the pumping rate and the permissible entrance velocity, and, in shallow aquifers, on the thickness of the aquifer. The diameter of the blind pipes in the screen section is usually the same as that of the screen diameter.

Example 22.5

To determine the total depth of a tubewell, we shall use the data of Example 22.2.

The length of the pump housing is based on the requirement that the pump remains below the water level inside the well. Suppose that the required design depth to the watertable is taken to be 2 m. The formation losses are added to this value. If the seasonal and long-term fluctuations of the watertable are estimated at 4 m and an additional length of 5 m is added for safety, the length of the pump housing becomes

$$2 + 5.6 + 4 + 5 = 17 \text{ m}$$

The screen length is primarily determined by the maximum screen entrance velocities. With a hydraulic conductivity of 25 m/d, this value, according to Table 22.3, is 0.015 m/s.

Suppose that a well screen is selected with an open area of 20% and a diameter of 0.25 m, we would then find the effective open area per metre screen length, bearing in mind a clogging percentage of 50%, to be

$$A_0 = 3.14 \times 0.25 \times 0.5 \times 0.20 = 0.08 \text{ m}^2/\text{m}$$

Substituting the above values into Equation 22.23 for a pumping capacity of 200 m^3/h = 4800 m^3/d yields

$$l_{min} = \frac{4800}{86400 \times 0.015 \times 0.08} = 47 \text{ m}$$

Assuming that the percentage of blind pipe to screen off unproductive layers of clay, silt, and very fine sand can be taken as 25%, the total length of the screen section becomes $47 \times 1.25 = 59$ m.

The total depth of the tubewell, together with a sand trap of 5 m, then becomes $17 + 59 + 5 = 81$ m.

Table 22.4 shows the minimum length of screen (section) and the total tubewell depth for screen diameters of 0.15, 0.20, and 0.25 m, and for different types of screens:
- Cheap well screens with an open area of 10%;
- Medium-priced well screens with an open area of 20%;
- Expensive, modern, continuously slotted well screens with an open area of up to 40%.

The total well depth in Column 6 consists of the pump house length (17 m), the screen length as calculated in Column 5, and the sand trap length (5 m).

Table 22.4 shows that tubewells with cheap screens (low percentages of open area) should be placed considerably deeper than tubewells with expensive screens (high percentages of open area). Taking into consideration the costs of drilling boreholes, it will be clear that wells with cheap screens are not necessarily cheaper to construct than wells with expensive screens.

The situation is more complicated with partially-penetrating tubewells. The deeper the well, the larger the penetration ratio, and the less the partial-penetration effect in the total drawdown inside the well will be.

In making the cost comparison, one should consider not only the construction, but also the costs of lifting the water to the land surface (i.e. the actual depths of the water level inside the well).

The last point to note is that Table 22.4 refers to a pump capacity of 200 m^3/h. Similar calculations should be made for the other two pump capacities (i.e. 100 and 300 m^3/h).

Table 22.4 Minimum screen lengths and total well depths for different types of screens (see Example 22.5)

Diameter (m)	Open area (%)	Open area (m^2/m)	Screen section Eq.22.23 (m)	Screen section + 25 % (m)	Total well depth (m)
(1)	(2)	(3)	(4)	(5)	(6)
0.15	10	0.024	157	197	219
0.15	20	0.047	79	98	120
0.15	40	0.094	39	49	71
0.20	10	0.031	118	147	169
0.20	20	0.063	59	74	96
0.20	40	0.126	29	37	59
0.25	10	0.039	94	118	140
0.25	20	0.079	47	59	81
0.25	40	0.157	24	30	51

Gravel Pack
The application of a gravel pack is recommended in the following formations:
- Fine sandy alluvium and aeolian sand aquifers;
- Alternating formations of fine, medium, and coarse sediment;
- Poorly cemented sandstone continuously losing fine material during pumping and giving no support to the screen because the formation does not fill up the angular space between the screen and the borehole wall supporting the screen immediately after the screen has been installed.

The selection of grading and grain size of the gravel pack depends on the sieve analysis of the finest layer included in the screen section in the productive part of the aquifer. Even finer portions of the aquifer are cased off with a blank pipe.

The rules for the proper design of the gravel pack, and for when to use single or double layers of different grading as gravel pack, will not be discussed here, but can be found in literature (e.g. in Huisman 1975 and Driscoll 1986).

Sand Trap
The sand trap is the section of blind pipe at the bottom of the screen section. Its function is to store sand and silt entering the well during pumping; this will occur even if the tubewell has been properly developed. The length of the sand trap is usually of the order of a few metres (2 – 6 m). The diameter of the sand trap is usually the same as that of the screen section.

Pump
The following factors determine the selection of the pump:
- The required discharge rate;
- The required head to be delivered by the pump. This head is made up of three parts:
  - The difference between the elevation of the discharge pipe into the drain and the natural surface level;
  - The water-level depth inside the pumped well, as discussed in Section 22.5.3;
  - Head losses due to friction and turbulence in the discharge pipelines between the pump and the drain;
- The efficiency of the pump;
- Pump durability. To keep maintenance and replacement costs to a minimum, the pump should be resistant to wear and to the corrosive action of the drainage water that will be pumped;
- In wells where the maximum water-level depth below the pump during pumping does not exceed 5 – 7 m, a suction pump, generally a centrifugal type of pump, can be used (Chapter 23). With deeper water levels during pumping, deep-well submersible pumps are required.

As will be discussed in Chapter 23, Section 23.2.2, the power requirement at the pump shaft is defined by

$$P_s = \frac{\rho\, g\, Q\, H}{\eta} \qquad (22.24)$$

where

P_s = power to be delivered to the shaft of the pump (W)
Q = pump discharge (m^3/s)
H = head delivered by the pump (m)
ρ = density of water (kg/m^3)
g = acceleration due to gravity (m/s^2)
η = pump efficiency (–)

Operating conditions being equal, the more efficient the pump, the lower the power requirements for pumping, and hence, the lower the pumping costs. Pump efficiency therefore becomes an important consideration when a pump is being selected because pumping costs usually play an important role in the economic viability of tubewell drainage.

Pump efficiency depends on the head-discharge relation and varies from one type of pump to another, and sometimes from manufacturer to manufacturer.

22.5.4 Design Optimization

The choice of the drainage method and the design of the drainage system are based on minimizing the cost of drainage. For tubewell drainage, this means properly selecting the well-field configuration and optimizing the borehole design so as to bring the groundwater to the land surface in the most economic way. Some optimizing options are:

- Water can be brought to the land surface by a well with a short screen or a long screen. A short screen involves low investment costs and high energy costs because of greater drawdowns in the wells, while a long screen entails relatively higher investment costs but lower energy costs (see Figure 22.9).
- The well can also have either a small-diameter screen (lower investment costs, high energy costs) or a large-diameter screen (high investment costs, low energy costs).

The optimization procedure involves examining the different well configurations that satisfy the design criteria, and, for each of these, calculating the investment costs and the annual costs of operation and maintenance. The present value of these costs is determined by applying an annual rate for discounting costs and an interest rate. The configuration that yields the lowest present value is then selected.

The steps to calculate the investments costs for a well with a pre-fixed pumping capacity and located in a given well-field lay-out can be summarized as follows:

- The well spacing and area of influence of each well is determined on the basis of the drainable surplus of the area in question, the discharge rate, and the maximum pumping hours per day;
- The minimum screen length is determined for the smallest screen diameter available and cheapest screen type, so consequently the lowest percentage of open area, in accordance with the criteria in Section 22.5.3;
- The pumping head, including drawdowns caused by the different factors discussed in Section 22.4.3, is calculated;

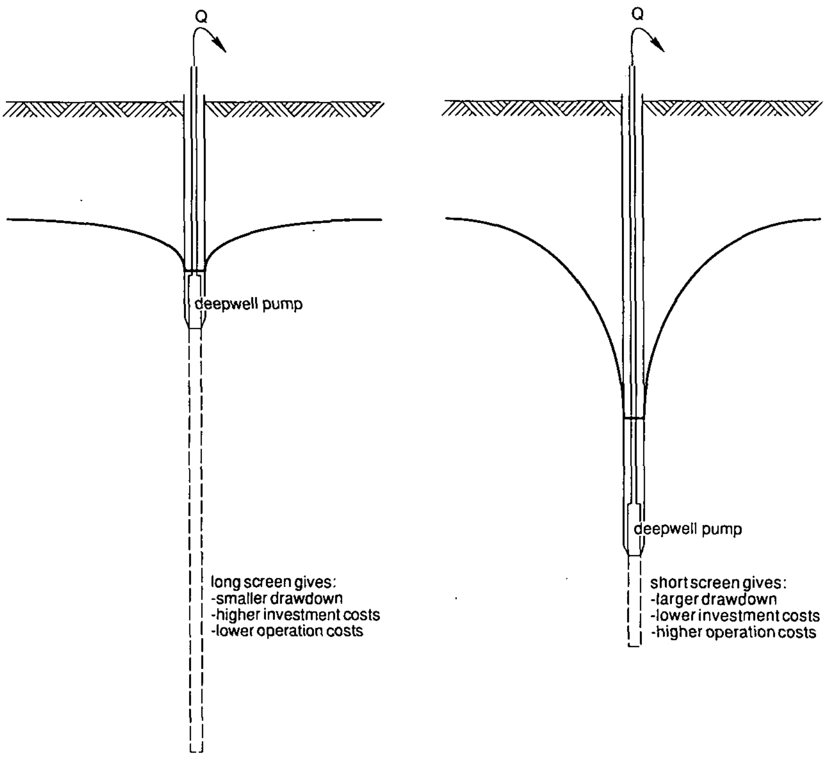

Figure 22.9 Alternative well designs regarding screen length

- The diameter and length of the pump housing, pump setting, and length of casings are selected according to pre-determined criteria;
- Drilling diameters are selected according to pre-determined criteria, and the volume of filter gravel is determined;
- The minimum motor capacity is determined and the smallest motor that exceeds this minimum capacity is selected;
- The investments costs (supply and installation) are calculated with unit rates;
- Other cost components, such as motor house, operator's quarters, costs of power distribution or fuel, and required water courses, are added to the investment costs;
- Annual pumping costs are calculated by totalling all annual costs, including investments, re-investments, energy, and the maintenance and operation of the well pump and engine and of the drainage system;
- Total costs are discounted and the present value is assessed by totalling the annual discounted costs. This value is converted into an annuity and an even cash flow. Finally, the annuity is divided by the quantity of water abstracted per annum, which results in the cost per m^3 drainage water or the costs to drain 1 ha;
- This procedure is repeated for longer screens, larger screen diameters, and other available energy types until the cheapest configuration is found.

Optimization means finding the tubewell design that drains the farmer's fields at the