15. ARTIFICIAL SUBSOIL DRAINAGE IN NEW SOUTH WALES

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ABSTRACT: *The design criteria for tile drainage depend on the need of horticultural plantings, which appear to require watertables deeper than 1.20 metres for average conditions. Design criteria tentatively determined for steady state formulas and local conditions control watertable depths to these levels, except during infrequent periods of excessive rainfall or, more frequently, excessive irrigation. This is confirmed by use of non steady state equations to fit a model of simulated watertable levels to observed values and subsequent extrapolation to more severe conditions, as occurred in 1956.*

The spatial variability in hydraulic conductivity, the infiltration characteristics and drainable porosity make the use of a safety margin in tile drainage design imperative. Measured hydraulic conductivity, which shows skew distribution, is not closely related to soil type. The proportion of total area in which drainage rates may be less than desirable may be assessed using the calculated variance.

Tubewell drainage has successfully been applied and costs of installation reduced by the introduction of P.V.C. screens to depths of 16 metres. Pumping from deeper aquifers frequently does not successfully control watertable levels because of vertical flow resistance through overburden materials. Boring for site selection is generally on a trial and error basis. The effect of pumping is determined by groundwater contour plotting of measured drawdown in observation wells. The effluent salinity frequently follows a pattern of increases, peaks, decreases to a final equilibrium, because of the nature of groundwater flow towards the pumping site.

Multiple stage drawdown testing is used to evaluate the well loss factor and for pumping unit selection purposes. Other methods of analysis to evaluate aquifer constants are discussed and found to be of limited value. The shallow aquifers in which tubewells are installed are frequently of limited areal extent, introducing anisotropy and hydraulic boundary problems, making predictions of the ultimate area of influence by the tubewell rather unreliable.

INTRODUCTION: The benefits of subsoil drainage are manifold; reduced watertable levels, better aeration, higher efficiency of nitrogen fertilizer, better traffickability and reduced salinity hazards resulting in increased production. These benefits have to be weighed against the cost of installation and operation of subsoil drainage systems.

Horizontal (tile) drainage has proved to be a very good investment in the horticultural areas around Griffith. Vertical drainage (tubewell) as applied in the horticultural areas around Leeton is more economical per unit area but not generally feasible in the absence of suitable aquifer systems. Vertical drainage has also been installed in non-horticultural areas subject to waterlogging and salinisation, such as in the Southern Riverina Irrigation Districts and the Tullakool, Curlwaa and Murrumbidgee Irrigation Areas. Both methods of drainage are described separately.

HORIZONTAL DRAINAGE: The theory of drainage of agricultural land and its application to practical situations has been described extensively (van Schilfgaarde 1974; Anon 1972-74). Maasland and Haskew (1957) describe the use of the augerhole method of measuring the hydraulic conductivity of soils and its application to tile drainage problems. Talsma and Haskew (1959) report on investigations of watertable response to tile drains in comparison with theory and found that the steady state approach worked satisfactorily. The Hooghoudt formula describing this condition is -

$$S^2 = 4km/q\ (2d + m)$$

where

S = drain spacing (m)
k = hydraulic conductivity (m/day)
q = drainage coefficient (m/day)
d = corrected depth of impermeable layer below drain lines (m)
m = height of watertable mid way between drains (m)

This formula is based on the Dupuit Forchheimer assumptions of horizontal flow between the watertable and the impermeable layer. Flow through the capillary fringe, which may be significant, is usually ignored. Convergence of streamlines near the drain is corrected for by adjusting the factor d. The adjustment is a function of S and r, the radius of the drain tile and usually small under M.I.A. conditions.

The drainage coefficient (q) was tentatively set at 5 mm/day with the watertable between the drains (m) at 0.45m from the soil surface (Maasland and Haskew 1958). This requirement was found to be met by actual installations (Talsma and Haskew 1959). The steady state conditions are related to the non steady state conditions via the drainable porosity (f). The first day after cessation of the steady rate effective rainfall equal to the design drainage coefficient (q) the watertable midway between drains will drop according to the ratio q/f (e.g. 10 cm when $q = 0.005$ m/day and $f = 0.05$). In heavier soils with smaller drainable porosity the watertable will drop more rapidly. For such soils a lower design discharge coefficient (q) would achieve the same drop of the watertable.

A lower drainage coefficient of 2.5 mm/day is being used for heavier textured soils. This implies that such soils do not allow an effective average infiltration in excess of 2.5 mm/day, the remaining excessive rainfall being carried away by surface drainage. The choice of the criterion in the M.I.A. depends on the infiltration characteristics of the surface horizons and the efficiency of the surface drainage system.

The approach followed is at variance with the overseas practice of increasing the steady state drainage coefficient as the drainable porosity decreases. This is based on the assumption that the effective infiltration does not vary with soil type (Bouwer 1974).

In field experiments the hydraulic conductivity (k) may be estimated by measuring the drain spacing (s) the drainage coefficient (q), the depth to the impermeable layer (d), and the height between two laterals (m), giving an average for the area of one drain spacing. The latter factor (m) is to be decreased with the height of the watertable level immediately above the drain tiles, which may occur for brief periods after rainfall or irrigation. Drainage design including submerged laterals is also feasible.

A number of non steady state solutions have been proposed, and several of them appear to describe actual field observations. Talsma and Haskew (1959) used the Glover formula in their investigations and found good agreement for moderated drawdowns. The tail recession of the watertable mid way between drains may be described as -

$$ht/ho = ce^{-t/j}$$

in which

ht/ho	=	relative height of the watertable on day t
c	=	coefficient = 1.16 if the initial shape of the watertable is a fourth degree parabola
j	=	reservoir coefficient = $f\ S^2 / \pi^2\ kD$ (days)
f	=	drainable porosity
D	=	effective depth of flow (m)
e	=	base natural logarithim

If the drain depth is 1.80 metres a drop in the watertable from 1.35 to 1.05 metres above drain level over four days would require a reservoir coefficient of ten days.

For typical conditions in the M.I.A. with drain spacings of 27 metres, f = 0.035, K = 0.20 m/day and D = 1.2 metres would be represented by a reservoir coefficient (j) of about 9 days. The reservoir coefficient represents the quantity of water stored in the soil between drains over the ability of soil layers to convey it to these drains. The watertable would recede to ht = 0.43 ho over the period described by the reservoir coefficient. It also represents an approximate period over which effective rainfall may be averaged to arrive at a satisfactory steady state drainage coefficient (q).

An important factor in non steady stage drainage evaluation is the drainable porosity (f). The drainable porosity relates to the quantity of water removed from the soil per unit drop of the watertable level. This quantity depends on the soil moisture characteristics and is smaller for very high watertables than for deeper watertables (Skaggs and Tang 1976; Bouwer 1974). The quantity drained per unit drop of the watertable also depends on the rate of drop and is not a fixed factor as commonly assumed. Across two drainage lines the drainable porosity varies as the depth to the watertable varies. Soil layering introduce further complications.

For the reasons stated above calculations with assumed constant average drainable porosity are subject to error. The watertable for instance may drop more rapidly than expected when it is high and slower when it is deeper. The variation demonstrates that application of drainage theory does not require a great deal of refinement since most factors involved can only be assessed by approximation.

The basis for design of tile drainage in the M.I.A. was determined during the fifties. This contribution discusses three areas not previously covered, viz., variability of hydraulic conductivity, performance of the design drainage coefficient and effluent salinity.

Variability of Hydraulic Conductivity

During the course of farm investigations for tile drainage about 15 000 measurements of hydraulic conductivity have been carried out using the augerhole method. In most instances the depth of the holes varied little, from 1.8 to 2.1 metres but the depth to the watertable at the time of measurement was either "normal", between 0.75 and 1.05 metres from the surface, or higher, or lower, or absent. A sample taken from all these measurements therefore is dependent on the condition encountered and it has to be assumed that the sampling was representative.

After sampling, comparisons were made between soil types, between farm portions of the same soil type, and with areas recommended for a certain drain spacing. The distribution of hydraulic conductivity is significantly skew at the one per cent level, as demonstrated in Figure 15.1. Conversion to a log basis removed the skewness which is in accordance with Nielsen *et. al.* (1973). Comparison of means therefore should be made on basis of the geometric mean, not the arithmetic mean. Table 15.1 shows the result of these comparison for 19 soil types of the Mirrool Irrigation Area.

The results of Table 15.1 show a significant difference between the geometric mean hydraulic conductivity of several soil types as mapped by Taylor and Hooper (1938). Individual measurements however vary considerably, the coefficient of variation of untransformed data within one soil type being 92 per cent.

A soil type mapping unit may be found on many farms or farm portions in the M.I.A. A significant difference of the mean hydraulic conductivity between farm portions was found and the mean coefficient of variation of measurements within one

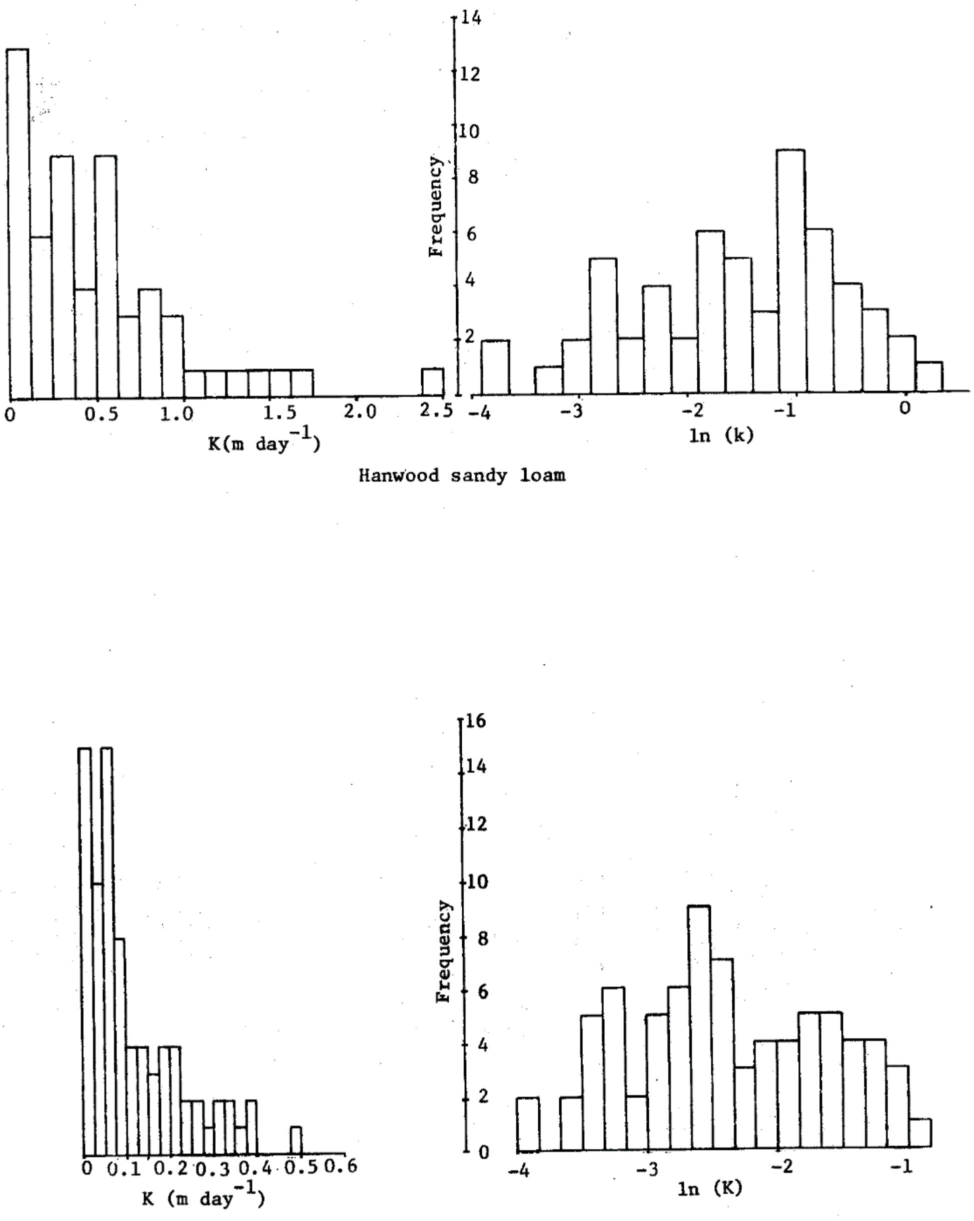

FIGURE 15.1 Frequency distribution of hydraulic conductivity of two soil types showing skewness of untransformed data (W.R.C. N.S.W.)

soil type on one farm portion was found to be 69 per cent.

In the M.I.A. subsoil drainage recommendations are not based on soil type boundaries. Within the area on a farm for which a common drain spacing is recommened there is still considerable variation of hydraulic conductivity. The co-efficient of variation was found to average 57 per cent.

TABLE 15.1 Differences between geometric means of hydraulic conductivity, normal watertables

Rank	Soil Type	Geometric Mean K (m day^{-1})	Geometric Mean Not Significantly* Different from:
1.	Ballingall loam	0.82	2, 3
2.	Tharbogang loam	0.64	1, 3, 4, 5
3.	Wyangan loam	0.48	1, 2, 4, 5, 6
4.	Hanwood loam	0.44	2, 3, 5
5.	Hanwood Sandy loam	0.34	2, 3, 4, 6, 7
6.	Yenda loam	0.30	3, 5, 7
7.	Bilbul loam	0.25	5, 6
8.	Griffith loam	0.18	9, 10, 11, 12
9.	Banna Sand	0.17	8, 10, 11, 12
10.	Hyandra Sandy loam	0.16	8, 9, 11, 12, 13
11.	Bilbul Clay loam	0.15	8, 9, 10, 12, 13
12.	Type 9	0.14	8, 9, 10, 11, 13, 14, 15, 16
13.	Yambil Sandy loam	0.12	10, 11, 12, 14, 15, 16, 17, 18
14.	Jondaryan loam	0.11	12, 13, 15, 16, 17, 18
15.	Yoogali loam	0.10	11, 12, 13, 14, 16, 17, 18
16.	Mirrool loam	0.10	12, 13, 14, 15, 17, 18
17.	Camarooka Sandy loam	0.09	13, 14, 15, 16, 18, 19
18.	Beelbangera Clay loam	0.09	13, 14, 15, 16, 17, 19
19.	Griffith Clay loam	0.07	17, 18

* at 5% level

No relation was found between the value of the hydraulic conductivity and the coefficient of variation as reported by Bouwer and Jackson (1974).

It was concluded that in the M.I.A. deep subsoil variability is not describable in terms of soil type boundaries. This contrasts with the practice in the Lower Murray region where drainage is installed at shallower depths and a relation between soil type and subsoil hydraulic conductivity may be reasonable. The values of k were collected for high, deep and normal watertables of each soil type. This enables calculation of averages of each soil layer where significant differences were found. Such information is useful in determining desirable drain depth but the variability described above has to be considered.

Design Criterion

The design criterion tentatively proposed by Maasland and Haskew (1957) proved to provide very adequate drainage and therefore was never modified, except for the correction for heavier textured soils with lower drainable porosity as already discussed.

Talsma and Haskew (1959) concluded that with the watertables mid-way between drains at 45 cm from the surface, the discharge rate agreed with the steady state design drainage coefficient. The number of occurrences when such high watertable conditions are reached however and the effect such occurrences have on growth of plantings are still subject to continued research efforts.

In some citrus varieties Minessy *et. al.* (1971) found a positive increase of yield if watertables are lowered to 175 cm. Penman (1938) observed that citrus trees remain healthy for the first 8-10 years of their life with a watertable within 1.20 metres of the soil surface. Beyond that age they require a deeper watertable. Balaam and Corbin (1962) surveyed all factors related to the yield of canning peaches in the M.I.A. and found that soil classification and watertable depth each contributed about 25-30 per cent of the total variation, with management practices accounting for a further 25 per cent. Positive relations were found between watertable depth during September and yield, and the data showed that the effect continues throughout the range of watertable depth to 2.10 metres.

Experiments as described above involve many dependent and independent variables which frequently are difficult to separate. For instance, salinity affects may be confused with effects of high watertables. From the results available it appears however that watertables in orchards should be kept as deep as possible and that conditions of high watertables should be controlled to brief periods.

At the Viticultural Research Station at Griffith (Turkington, pers. comm) an experiment has been carried out using various drain spacings and plants to test the design criteria. During average conditions the watertable was found to fluctuate between 1.20 and 2.10 metres without a deleterious effect on plantings in any treatment. Only after deliberate ponding and subsequent rainfall did some damage in peach plantings occur, mainly in the "wider than recommended" spacing treatment, and also in the narrower spacing treatments. In the latter instance, surface waterlogging was thought to have been the cause rather than slowness of watertable recession. It was concluded that the criteria presently used are adequate. Watertables should drop from 45 cm to 75 cm below the soil surface over a three day period.

The non steady state theory allows calculation of watertables between drains on a day to day basis. Such calculations may be compared with actual watertable behaviour and some parameters modified to obtain a good fit. Figure 15.2 shows the effect of wet conditions during 1973 on watertable behaviour at Farm 77, near Griffith. Rainfall runoff at this farm is known to be low. Initial infiltration is used to reduce the soil moisture deficit which is calculated on a day to day basis using the evaporation of the class A pan multiplied by a factor 0.5. A drainable porosity of 0.04 was assumed on the basis of hydraulic conductivity measurements. Water reaches the watertable only if the soil moisture deficit has become negative. The agreement between observed and calculated lines is considered reasonably good. It is shown that during the autumn and winter of 1973 on this farm the watertables averaged about 1.30 metres below the soil surface. The reservoir coefficient providing the best fit was 12.5 which agrees with the design criteria used on this farm. The same winter about 40 per cent of peaches in the Goulburn valley and non tile drained land of the M.I.A. were seriously damaged due to waterlogging.

Having calculated a reasonable fit with the observed watertable behaviour the computer program may be used to calculate the watertables during the wettest year on record, which is 1956. It was found that average groundwater conditions on this farm would have been at about 80 cm from the surface had drainage been installed. During late autumn and the winter the watertable came to within 45 cm from the surface on two occasions, both of which lasted less than three days.

The variability of observed watertables was rather large, the standard deviation of the mean being about seven cm. The largest variation is found after irrigation when some parts of the farm may receive more water than others. The variability in hydraulic conductivity causes some areas to be drained more rapidly than others. Skaggs and Tang (1976) calculated watertable drawdown for hydraulic conductivity values plus or minus one standard deviation and found a large difference in time required to obtain a 30 cm drop. The variation of k as found in the M.I.A. may be used to make assessments about the drainage efficiency. Since part of the land for which a drainage spacing is calculated has an hydraulic conductivity less than the mean this part will drain at a rate of discharge less than the design discharge coefficient (q). Table 15.2 shows details of the proportion of land draining at lower than design discharge for various ratios of the arithmetic mean over the actual mean of hydraulic conductivity.

It shows that if the design is based on the geometric mean hydraulic conductivity the area of land draining at a rate less than the design discharge coefficient is 40 per cent. With

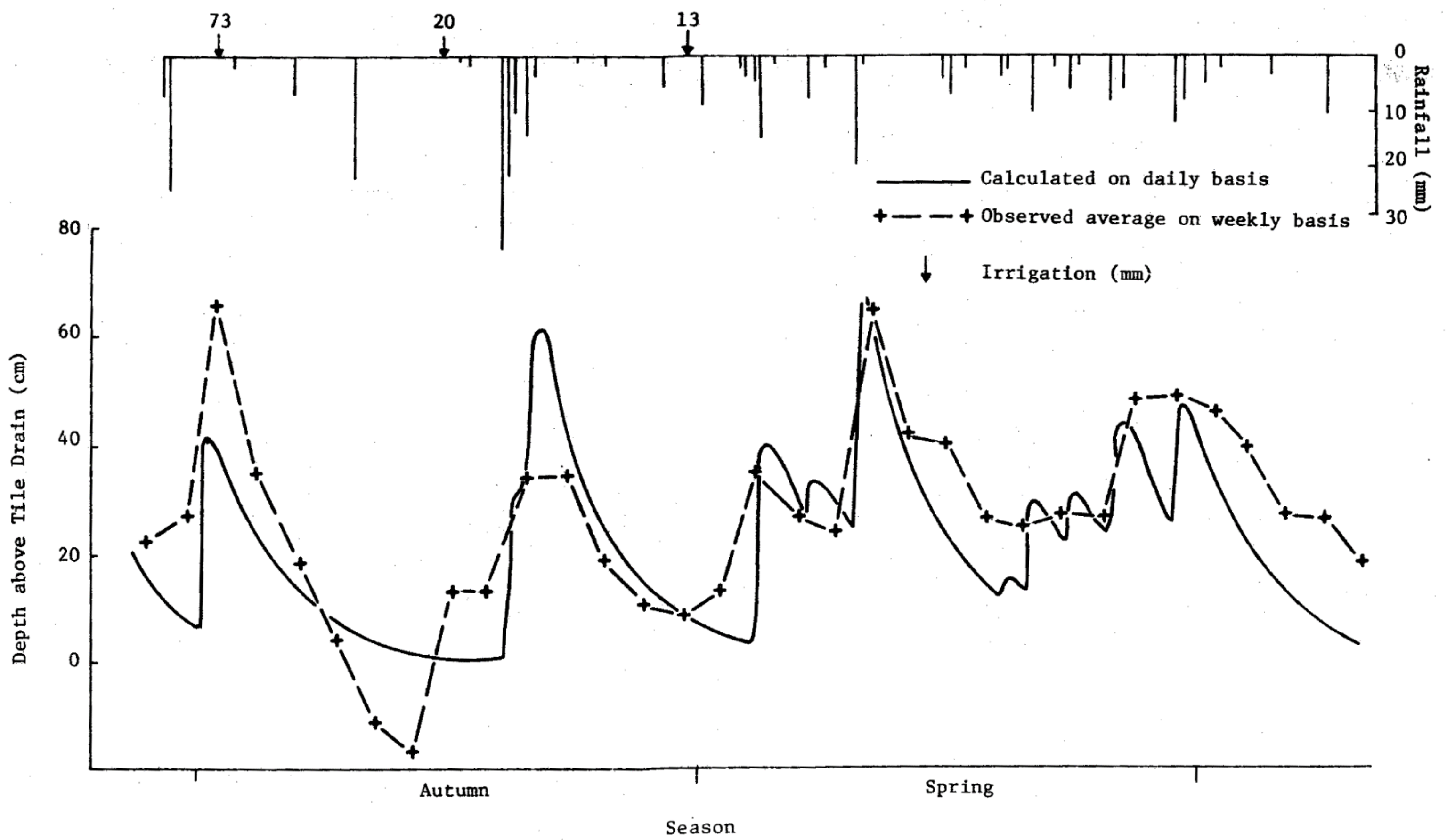

FIGURE 15.2 Calculated and observed watertable behaviour at Farm 77 during 1973. —— calculated on daily basis; —+— observed average on weekly basis (W.R.C. N.S.W.)

nine per cent of the recommended areas the proportion of land draining at less than the design drainage coefficient is 69 per cent or 56 per cent for the arithmetic mean or geometric mean hydraulic conductivities respectively.

It is concluded that drainage installations designed using the present criteria control watertables at relatively deep levels, but a large variation is found. As far as rainfall events are concerned the average watertable would rise to levels corresponding with the design criteria or higher only on rare occasions.

TABLE 15.2 Probability range of ratio arithmetic mean/actual mean hydraulic conductivity and percentage of areas draining at design discharge or less

Arithmetic Mean / Actual Mean	Probability that Ratio of (1) is exceeded*	Percentage of Area Draining at Design discharge q or less	
		Design with Arithmetic Mean	Design with Geometric Mean
(1)	(2)	(3)	(4)
0.7	99%	30%	24%
0.8	93	36	29
0.9	75	43	34
1.0	50	50	40
1.1	29	57	46
1.2	16	64	52
1.3	9	69	56
1.5	2	82	69

* based on 12 available observations and a coefficient of variation of 57%

Effluent Salinity

Investigations on seven farms over a number of years were carried out to study rainfall, irrigation, drainage discharge and salinity relationships. On average 16 per cent of irrigation or rainfall was discharged through the tile drainage system, much of it as a base flow after temporary storage as groundwater. No relationships between discharge, rainfall or irrigation, and salinity was found but it did appear that the highest salinities are related to fall of the watertable, rather than its height. The shape of the salinity - time curve, although subject to large variations, appears to decrease linearly with time. A survey over a large number of drainage installations of varying age showed that it would take about

25 years before the ultimate equilibrium salinity is reached. The decrease found is described by the regression

$$EC = 5980 - 255T \; (^{**}r = 0.39, df = 65, sd = 2210);$$

where EC (electrical conductivity) in microSiemens cm^{-1} at $25^{o}C$ and T in years

Increased rainfall or irrigation increased the salt load discharge by the tile drainage pump. The total load discharged by about 400 pumps was estimated at 25 000 tons for 1972. A reduced rate of installation of drainage in subsequent years and the above regression gave the tentative result that the salt load from the drains may be reduced considerably after 1980. The quality of the effluent is not good enough for direct re-use but dilution of the effluent with irrigation run-off and channel escape water is such that at the lower end of the surface drainage system the quality of the water is re-usable for irrigation both from the salinity and other quality aspects points of view.

Practical Application

The method of investigation for routine tile drainage design is described by Maasland and Haskew (1958). The density of holes is five per two hectares. After plotting relevant information on aerial photographs areas of similar hydraulic conductivity values are delineated. These areas do not usually correspond with soil type boundaries and within such defined areas the coefficient of variation is less that for soil types as a whole. The number of measurements within each such area determines the confidence limits of the mean. With a standard deviation of individual measurements equal to 57 per cent of the mean a minimum of 12 measurements would be needed to obtain a standard deviation of the mean of 16.5 per cent of the mean. In many cases a smaller number of measurements are available, reducing accuracy. It is however not considered worthwhile to increase the number of measurements for these small sub-areas as the cost of each measurement at present being in the order of $15.00.

After design and survey machines install drainge laterals which discharge through inspection pits into main lines towards the pumping sump. The capacity of the pump is determined with the steady state discharge rate expected when watertables between drainage laterals are at 0.45 m from the soil surface. Non steady state equations would give a much higher discharge rate the first few days after raising the watertables to these levels. These conditions however are reached only very rarely during the autumn and winter periods and the existing pump capacity is considered adequate. Some problems occasionally occur during the irrigation season when irrigation is followed by rainfall and the pump capacity is the limiting factor in drainage, causing some backing up of water in the tile drainage systems.

The highest peaks in observed discharge occur during the irrigation season, not the late autumn and winter periods, for which the system is primarily designed.

VERTICAL DRAINAGE: Pumping from tubewells installed in sandy aquifers induces a cone of depression in pressure levels to a certain distance from the pump site. The distance depends apart from the pumping rate on the transmissivity of the aquifer and, if the aquifer is not confined, on the recharge from shallower layers. The recharge from the shallower layers depends on the thickness (d) and vertical hydraulic conductivity (k), termed the vertical resistance (c = d/k). Without recharge from the soil surface the watertable will drop and eventually become equal to the pressure level in the aquifer. This condition was encountered near the Hanwood pump site whilst pump testing during the dry winter of 1976. Drawdowns with time of both pressure level and watertable with time are shown at Figure 15.3.

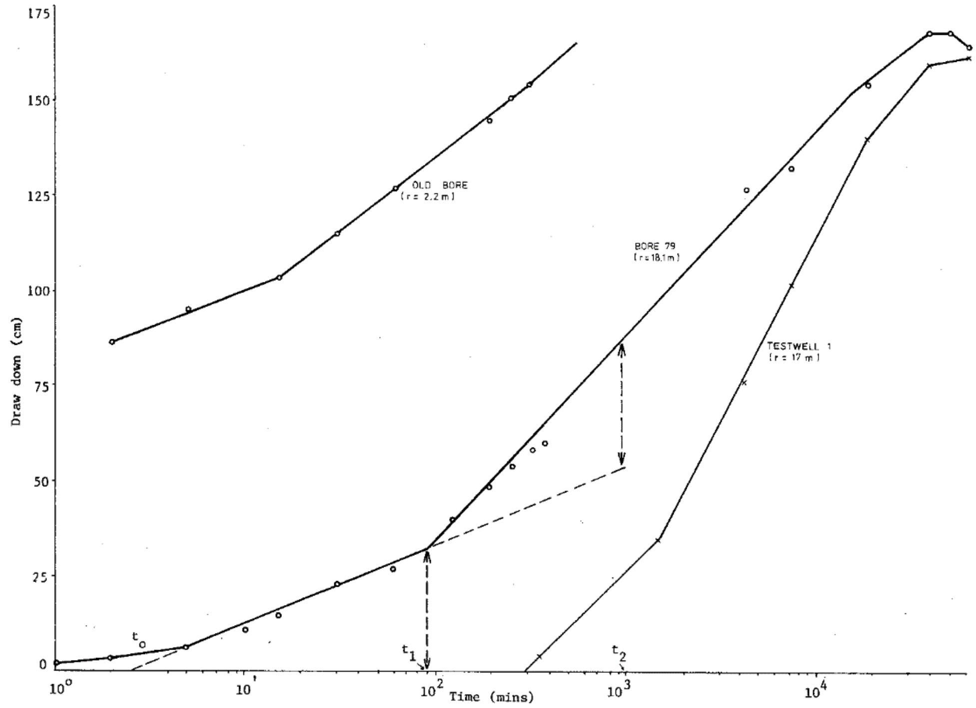

FIGURE 15.3 The effect of time on drawdown in wells near Hanwood (W.R.C. N.S.W.)

During irrigation or rainfall some recharge to the watertable usually takes place. The cone of depression will not expand after the recharge over an area surrounding the tubewell and its discharge rate have become equal. Since the recharge varies with time, the area influenced by the tubewell varies with time. The effective area of influence is to be related to the worst conditions encountered, e.g. a once in ten year rainfall event. The recharge to the aquifer also varies with the location's surface runoff and infiltration characteristics. Each tubewell therefore has to be evaluated on its effective area of influence separately.

There are 15 tubewells installed in the M.I.A. and a larger number in the Southern Riverina Irrigation Districts and Lower Murray Irrigation Areas to control watertable levels. Turnell (1976) describes technical details and conditions pertaining in the M.I.A. The cost of construction of these tubewells in aquifers to 16 metres deep has decreased in recent years because of the introduction of P.V.C. replacing metal casings and screens. Additional benefits are reduced well losses and higher resistance against corrosion. Small sized gravel envelopes are normally used to reduce radial flow resistance near the screen. These P.V.C. tubewells may be installed with a Gemco drill which works faster than the previously used percussion drills. A typical bore would have a 20 cm diameter P.V.C. screen with a 5 cm thick gravel envelope having a length of 2.5-3.5 metres, penetrating the aquifer over about half of its total thickness.

It has been the experience that watertable control can be achieved most efficiently when the tubewell is installed in a relatively shallow aquifer, based at less than 16 metres. With deeper aquifers the cone of pressure level reduction may be larger but the effect on watertables levels frequently is difficult to detect because of an excessive vertical resistance for flow through overlying materials. The Yoogali bore, near Griffith installed at 30 metres below the soil surface was abandoned after two years of operation for this reason.

Investigational Procedure

The present procedure used for locating sites suitable for tubewell drainage is as follows:

1. *Boring to establish stratigraphy and extent of the aquifer* - This may be preceded by electrical resistivity surveys but results of such surveys frequently are difficult to interpret. Shallow clay layers saturated with relatively salty groundwater have very low resistivity. High salinity of groundwater in the aquifer obscure the sands in this method. However it is not uncommon for a major underground stream to contain less salty groundwater, allowing it to be recognised by the resistivity method.

2. *Pump testing of pilot bores* - A relatively short, small diameter screen is lowered to the bottom of the aquifer and pumped at the maximum rate for a brief period. The screen is then lifted over its length and the procedure repeated. The pumping rates achieved by experience indicates the suitability of the site and the layer within the aquifer with the highest permeability is found. The salinity of groundwater from the various layers is measured.

3. *Availability of disposal facilities* - Pumping into irrigation supply channels has not been practised to date but has been commenced on an experimental basis in Berriquin Irrigation District in locations where groundwater salinity is not very high.

4. *Availability of power, single phase or three phase* - The amount of power available puts a limit to the size of the bore and selection of pumping equipment.

5. *Accessability of the site, for servicing etc.* - Items, 3, 4 and 5 need consideration at an early stage to avoid escalation of costs.

6. *Selection of site and pump testing* - An appropriate screen is installed at the site selected and a more elaborate pump test carried out to determine the required pump capacity, to predict the lateral extent of the cone of pressure reduction and to evaluate the transmissivity (T), the storativity (S) and the well loss factor (C). Observation wells are installed in the vicinity of the pumping site to provide the data required in the analysis. This testing may continue for several months. The range of values of T and S found in N.S.W. irrigation areas and districts are shown in Table 15.3.

TABLE 15.3 Range of transmissivity and storativity found for pump test sites in N.S.W. irrigation areas

Type of Aquifer	Transmissivity m^2 day^{-1}	Storativity (%)	Number of Sites
Ancestral River	1043 - 4655	0.17 - 4.0	2
Prior stream	561 - 2255	0.40 - 4.0	6
Minor prior stream	401	0.02	1
Floodplain aquifer	80 - 1428	0.02 - 0.81	7

N.B. A tubewell is usually not installed at sites with T less than about 600 m^2 day^{-1}.

The alternative of installing a number of tubewells close together is sometimes considered, especially with very shallow aquifers in which insufficient drawdown is available. These tubewells are joined to one pumping unit, as with spear point systems.

7. *Selection of pump* - Three types of pumps are used, vertical turbine, electro-submersible and centrifugal, the latter type only being suitable for moderate drawdowns. The cost of pumping of effluent is similar for all types, in the order of \$3.50-\$4.00/megalitre. Pumping rates from existing tubewells vary between 9 and 30 l sec^{-1}.

8. *Area of pressure relief* - About six to nine months of operation for units discharging in the order of 20 l sec^{-1} are needed before equilibrium is obtained. A network of observation wells from which watertables are read allows plotting of groundwater contour maps to evaluate the effectiveness of the pump. The cost of these investigations is relatively high.

Analysis of Pumping Test Data

Two types of pumping tests may be distinguished, well tests and aquifer tests. The first type usually is a multiple stage drawdown test to evaluate characteristics of the bore. The aquifer tests give values for aquifer constants, such as transmissivity. Both types of tests are required for proper evaluation. The conditions during the pumping may be steady state or non steady state, carried out in confined, semi-confined or unconfined aquifers. Mostly water is being pumped at a certain fixed rate and the drawdown in the pump well and/or observation wells at various distances is measured. Such observations may be continued until a steady state situation is reached which may be a couple of months. The data are then used for analysis.

Most methods of analysis revolve around a number of assumptions which for shallow aquifers in the Riverine Plain are not usually satisfied. The analysis therefore is mainly by approximation. The results are in terms of well loss factor via the multiple stage drawdown test, the transmissivity of the aquifer, the storage coefficient and, with semi-confined, leaky aquifers, the resistance to vertical flow through the (clay) overburden. Comparison of these results between sites may be subject to error if the basic assumptions underlying the method of analysis do not comply. For a discussion of pumping test analysis and theory reference is made to Kruseman and de Ridder (1970), de Wiest (1965), de Ridder (1973), Wesseling and Kruseman (1974) and Wisler and Brater (1959).

Multiple stage drawdown testing is a convenient tool to evaluate the well loss factor (c), the loss of head induced by turbulent radial flow near the well. During brief periods of pumping at increasing rates the increase in drawdown (ds_i) per increase in discharge (dq_i) is assessed when the drawdown approaches an apparent equilibrium. The well loss factor is -

$$C = (ds_i/dq_i - ds_{i-1}/dq_{i-1})/(dq_i + dq_{i-1}) \text{ sec}^2 \text{ m}^{-5}$$

(Rorabuach, 1953) - (1)

Values of this factor found range from about 600 for a number of tubewells near Leeton (Woodyer 1956) to about 3300 at Simpson's lane in the Tullakool Irrigation Area and about 36 000 at one Coleambally townsite bore. Walton (1962) considers well loss factor of about 3800 and larger to be unsatisfactorily high. No comparative data are as yet available for the difference in performance between the previously used metal screens and the P.V.C. gravel packed screens used at present. At the South Murrami P.V.C. tubewell site a value of C = 1000 sec^2/m^5 was found.

The drawdown in the tubewell, or any of the observation wells installed, with time depends on the conditions within the aquifer. Apart from the degree of confinement, the drawdown is affected by anisotropy in the aquifer, boundary effects and the slope of the aquifer, the latter factor usually being of lesser significance in the Riverine Plain in the calculation of aquifer constants.

Anisotropy may be confirmed by calculation of transmissivity (T) using observation wells in different directions away from the pumping site. It may indicate the direction of deposition of the coarsest sediments within the prior stream and the relative suitability of the pumping site for use as a permanent site.

Hydraulic boundaries are the reason for significant errors in the calculation of transmissivity and related factors and this factor needs to be evaluated. The first step in the analysis of available data should be the plotting of the drawdown and log-time data for a couple of wells close to the pumping site. It is important to obtain data for times very soon after pumping commences to several hours after. For this a single stage drawdown test lasting a full day may be necessary.

Various methods to analyse boundary effects are listed by Kruseman and de Ridder (1970). The simplified approach by Wisler and Brater (1959) is discussed here.

Jacob's formulas (Cooper and Jacob 1946) for drawdown (ds) and storativity (S) in confined aquifers or unconfined aquifers with relatively low ratio drawdown/thickness aquifer are:

$$ds = 2.3\ Q/4\pi T \quad (2)$$

$$S = 2.25T\ to/r^2 \quad (3)$$

provided the condition

$$u = r^2 S/4Tt < 0.01 \quad (4)$$

is satisfied. This applies for small distance (r) from the pump site and larger times (t). The drawdown verus log t then is a straight line with one log cycle of time determining the value of ds. Using the pumping rate (Q) the transmissivity (T) and the storativity (S) may be calculated using the intersect of the straight portion of the line with the time axis.

If a hydraulic boundary occurs at not too great a distance, its effect will show up in an increased rate of drawdown. In the case of a single straight boundary the slope of the line on a drawdown versus log-time plot would double. If the effect comes in very rapidly the lower portion of the original drawdown line may be obscured and the larger slope would be used for the calculation of the transmissivity, which would then only have half the value. An example is shown in Figure 15.3 for the data of the Hanwood pump, near Griffith. The data for bore 79, at distance r = 18.1 metres for times less than 5 minutes do not satisfy equation (4) and are ignored. The straight section between times t = 5 and t = 90 minutes yields with equations (2) and (3) T = 620 m^2/day and S = 0.008. After this the slope of the line doubles as a consequence of an hydraulic boundary.

The drawdown increase may be represented as being caused by the pumping well and an image well on the other side of the boundary. The distance r_2 of the observation well to the image well can be calculated by

$$t_1/r_1^2 = t_2/r_2^2 \quad (5)$$

provided equation (4) is satisfied and r_1 is the distance to the pumping well, and t_1 and t_2 are times necessary to bring about a certain equal drawdown. From Figure 15.3 it is found that at the Hanwood pump site an image well would occur at 59 metres from Bore 79. This calculation does not show in which direction this boundary would occur. Analysis from more than one observation well would be needed but the data of the "old bore" or wells at greater distance from the pumping site do not allow confirmation of the result. Boring revealed however that a boundary exists at the Hanwood pump at less than 60 metres in a north westerly direction.

The existence of impervious boundaries may upset the analysis, the drawdown resulting from pumping frequently being larger than the drawdown which would result in an infinite aquifer. Multiple stage pump testing at the South Murrami site showed a well loss factor of (C) 2548 $sec^2\ m^{-5}$ for steps of 24 hours and only 1006 for steps of 2 hours. At times larger than about one hour, rates of drawdown doubled, probably because of the existence of a hydraulic boundary. This would increase the value of the well loss factor C (see equation 1). It is to be noted here that the low values for the well loss factor of Woodyer (1956) were obtained with one hour steps in the pumping rate.

Similarly with linear aquifers the drawdown in observation wells at distance l from the pumpsite will be higher than the drawdown at the same distance in infinite aquifers, all other factors being equal. Close to the pumpsite and at small times (t) the groundwater flow may be radial and the theory for infinite aquifers would possibly apply.

The various methods of pumping test data analysis need a brief comment. The non steady state and steady state formula for semi-confined aquifers apply where a watertable or ponded water on the surface is maintained and the downward flow to the aquifer is proportional to the reduction of the pressure level in the aquifer. Therefore there is a decreasing replenishment with increasing distance from the pumping site. This condition only applies if the tubewell is surrounded by rice fields or waterlogged swamps, or if aquifers shallower than the pumped aquifer redistribute shallow groundwater. The latter condition does not usually occur with drainage by tubewells in the N.S.W. Irrigation Areas and Districts.

Mostly the watertable will drop as the pressure level is reduced, frequently with a time-lag, as shown in Figure 15.3 for test well 1 near Bore 79. The drop of the watertable suggests the use of Boulton's delayed yield method for unconfined aquifers. At the Hanwood pump, however, Boulton's (1964) method does not apply and the Theis (1935) equation may be used. The low value of the storativity S = 0.8 per cent calculated for small times, is related to the compressability of the materials as well as to the drainable porosity at watertable level. This latter factor is believed to be in the order of three per cent since the hydraulic conductivity of the overburden materials is about 0.12 m day^{-1} (Talsma and Haskew 1959).

The purpose of tubewell pumping is to effect the largest possible area of extent of pressure level reduction. The only reliable method of determining this area is by groundwater contour mapping. It is an expensive item of tubewell drainage as many observation wells are required and frequent readings are necessary.

The area protected is related to the drawdown achieved at a distance from the tubewell, sufficient to cope with the requirement. The drawdown required to achieve a drainage coefficient (q) is equal to cq, with c the vertical resistance for flow between the watertable and the aquifer. For a c value of 100 days the drawdown required to achieve q = 2.5 mm day^{-1} for instance would be 0.25 metres.

If the aquifer is replenished at a steady rate (q) the effective radius of influence by the tubewell is $r^2 = Q/\pi q$; Q being the pumping rate. From groundwater contour analysis it appears that q varies from between 1.2 and 3.2 mm day^{-1} for the Leeton District tubewells to as little as 0.5 mm day^{-1} at the Fullers lane site, near Finley. At the latter site irrigation intensity is lower, resulting in lesser rates of replenishment.

Effluent Salinity

The hydraulic gradient towards the tubewell causes salts from various pockets within the complex aquifer system to become dislodged. Significant changes in the distribution of salt both within the aquifer and within the aquiclude may be expected from withdrawal and replenishment of the groundwater. A long term equilibrium would develop dependent on the physical properties of the aquifer, the salinity distribution within the sphere of influence, both horizontally and vertically, the salinity distribution of the aquiclude overlying the zone of influence and the conditions under which recharge and leaching to the watertable occurs.

Tubewells are usually installed in the parts of the aquifer having the highest transmissivity. At these sites the salinity frequently is lower than in some more isolated pockets of the same aquifer as the clay overburden is less restrictive to deep percolation or may even be absent. After commencement of pumping an increase in salinity of the effluent may be observed as water is drawn from locations further away from the tubewell and salts start to enter the aquifer from the aquiclude. The effluent salinity then stabilises until after a long period the leachates start to become depleted, first at short distances from the tubewell and later at increasing distances. The final equilibrium would depend on the irrigation/rainfall input and mineral weathering from the aquiclude and some leakage from beyond the zone of influence of the installation.

All tubewells may be classified depending on the salinity pattern the tubewell displays. Around Leeton three tubewells show increasing salinity, three have an apparently stabilised peak salinity, three display a decreasing salinity and from two tubewells the salinity appears to have reached the low level equilibrium. For the tubewells in the M.I.A. the salinity of the effluent varies from 1000 - 4000 microSiemens cm^{-1} while in the Southern Riverina Irrigation Districts it exceeds 20 000 microSiemens cm^{-1}.

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