On the Analysis of Dewatering Systems

Analyse des Systèmes d'Epuisement

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SYNOPSIS The demand for greater analytical support for dewatering system design is identified, and specific areas are discussed with reference to wellpoint and deep well methods. These areas include the effect of partial penetration, the estimation of the boundary conditions and the influence of sheet piles.

INTRODUCTION

The objective in the design of a dewatering scheme is the specification of equipment in terms of wells and pumps. The specification is derived by means of a geotechnical analysis which yields the number and type of wells, their location and depth, and their flow capacity. A subsequent assessment is then made of the number and type of pumps and ancillary plant that will be required. The purpose of this paper is to draw attention to the absence of efficient rational design methods for certain commonly found types of dewatering schemes.

There are of course a group of dewatering situations that are so common that they can be designed almost by rule of thumb as soon as a site investigation has confirmed that no special complications are likely to arise. For example, it is not difficult to specify for trenches or rectangular excavations up to depths of 6m in homogeneous unconfined aquifers of modest depth with permeabilities in the drainage range. Reference to the excellent chapter on dewatering by Mansur and Kaufman (1962) in Leonards' "Foundation Engineering" is very fruitful in these cases.
However as soon as the foundation depth exceeds 6m, and the aquifer depth exceeds the foundation depth by a substantial amount, the problems of design move outside the scope of a rule of thumb approach and appear to require the application of theories which are more difficult to apply by an order of magnitude. As a result a completely rational design can become very expensive.

In the majority of foundation works which require dewatering the contractor is usually insistent that such a design is carried out so that there is some assurance that the dewatering proposals will work first time. Delays through failure to lower groundwater levels adequately or through heave due to insufficient pressure relief, occuring at such an early stage in the contract, are usually very expensive to cope with. Reliability rather than cost is the main concern and reliability can only come through experience backed up by a sound theoretical approach. There is rarely room for a "suck it and see" approach, unless the job is so large that pumping tests and modifications to the dewatering system can be made as the job progresses.

It is possible to identify three main stages in the evaluation of a proposed dewatering scheme. These stages are, (1) the estimation of where the boundaries of the dewatered region are located, and these boundaries include the positions and depth of the wells, (2) the calculation of piezometric surfaces and pore water pressures that will occur within the assumed boundaries, and (3) the assessment of aquifer properties and water chemistry for the purpose of estimating yields and anticipating long term corrosion problems. Each stage is of course carried out bearing in mind the constraints imposed by the proposed works and the information obtained from a site investigation report.

These stages will now be considered in turn and some special analytical difficulties that currently present themselves will be identified and commented upon.

BOUNDARY CONDITIONS

Two boundary conditions which have an important influence on design are often particularly difficult to establish. One is the position of the effective depth of the aquifer being dewatered and the other is the location of the boundary of the region outside of which the dewatering has had no influence. For a single well this latter boundary is often called the radius of influence.

Considering each in turn,

Aquifer depth

This boundary condition locates the depth to which the permeable strata below the foundation is likely to be affected by the dewatering wells. This depth is often limited by the occurrence of a substantial impermeable strata of clay or bedrock. However in many cases the existence of such a boundary is not confirmed by site investigation information, especially in deep aquifers, and the position of the boundary has to be assumed.

The occurrence of a deep aquifer inevitably leads to the problem of determining piezometric levels and flows for assemblies of wells which only partially penetrate the aquifer and for which design procedures are limited by their complexity. Although some attention has been given to single partially penetrating wells, largely for the purpose of designing wells for water supply, little is known about the accuracy of superposition procedures for the groups of wells which occur in the context of dewatering. The problem is not so much the prediction of flows, as the approximations that can be made for this purpose tend to lead to an over-estimation of yield. It is in the prediction of piezometric levels that the difficulty lies.

Some guidance on this is given by Chapman (1956) for the case of two dimensional slots, and his formulae are suitable for use in the design of partially penetrating wellpoint systems. However this information appears to be limited to the case where the ratio of the distance to the boundary of influence to aquifer depth is equal to approximately three. The information is in the form of two simple formulae which relate flow per unit width q/x and centre of excavation piezometric level h_D, to aquifer depth H, depth of penetration h_o, distance to boundary of influence L, and permeability k. Thus,

$$q/x = k\left(0.73 + 0.27\,\frac{H-h_o}{H}\right)\frac{(H^2 - h_o^2)}{2L}, \quad \frac{L}{H} = 3. \qquad (1)$$

$$h_D - h_o = h_o\, C_1\, C_2\, (H - h_o)/L, \quad \frac{L}{H} = 3. \qquad (2)$$

Figure 1 is a section through a three stage wellpoint dewatering system that was used for an excavation for a pumping station in Nigeria. The figure defines the parameters used in equations (1) and (2) and demonstrates the requirement for partical penetration formulae. The coefficients C_1 and C_2 in equation (2) are functions of the excavation and dewatering system geometry and are given in Chapman's paper or can be found in Mansur and Kaufman (1962).

In order to relieve these formula of the L/H constraint an attempt has been made to calibrate them for other values of L/H by use of analogue models of a range of typical dewatering geometries. Initial results indicate that the simplicity of Chapman's formulae, which is a very attractive feature from the designers point of view, can be preserved and the range of L/H extended by writing them as

$$q/x = k\left(g + (1 - g)\,\frac{(H - h_o)}{H}\right)\left(\frac{H^2 - h_o^2}{2L}\right) \qquad (3)$$

and

$$h_D - h_o = h_o\, C_1\, C_2\, C_3\, (H - h_o)/L \qquad (4)$$

where g and C_3 are functions of L/H given in figures 2 and 3.

The values of g for L/H<3 are tentative for the study is incomplete, and the analogue results for h_D, although correlated following Chapman are in slight disagreement with his conclusions.

A widening of the applicability of these simple design formulae in this manner would be very helpful, and it is hoped that the work described here will be completed for wellpoints and then extended to include partially penetrating tube wells.

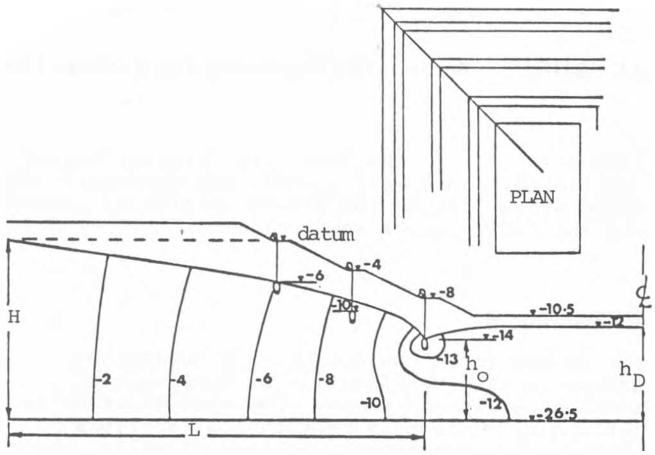

Fig.1 Wellpoint design for a Nigerian pumping station.
Levels in metres, third stage operating.

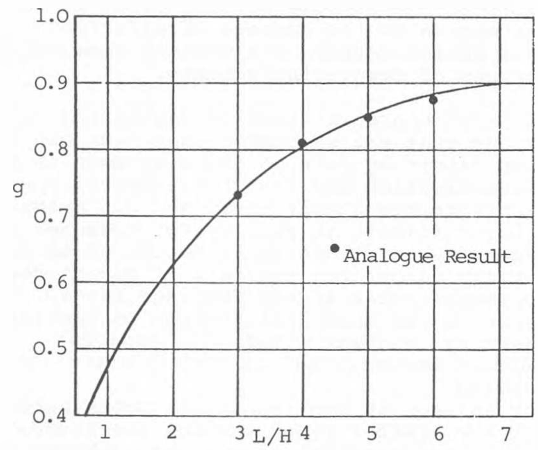

Fig.2 g(L/H) in equation 3

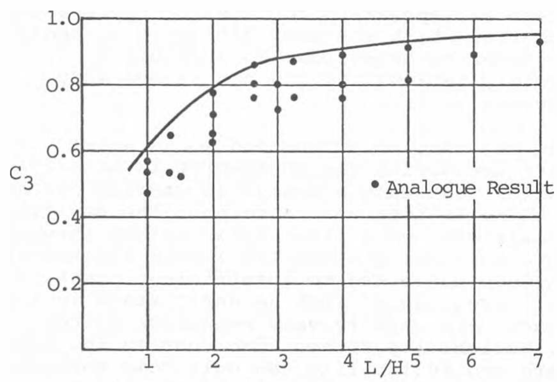

Fig.3 C_3(L/H) in equation 4

Boundary of influence

It is evident that in the design of any dewatering system it is necessary to locate this boundary. In equations (1) to (4) for example the choice of the value of L is obviously important in assessing the design of a scheme such as that illustrated in figure 1. The difficulty here is that the position of the boundary is time dependent. Shortly after pumping has commenced it is very close to the excavation but is receding quite rapidly. Hydraulic gradients are large and high flows would have to be handled to produce the required drawdown. Some days, maybe months, later it will have receded 1000m - 2000m from the wells and will have almost stabilised with flows also becoming stable and relatively lower. This feature can be used to advantage in the specification of the number of pumps required for a particular job. It is usual practice to have one pump standing by for every two working pumps. During the initial stages of dewatering, when flows are high, the standby pumps can be brought into temporary service in order to establish satisfactory working conditions at a relatively early stage. These pumps can later be reserved for standby duty once the yield from the scheme has reduced sufficiently.

A simple formula for estimating the radius of incluence, R, for tube wells, and which may therefore be helpful in indicating the sort of values that should be used for L in equations (1) to (4), is Sichardt's formula, Mansur and Kaufman (1962). This formula is

$$R = C(H - L_o)\ \sqrt{k} \qquad (5)$$

where R is given in metres if k is the permeability in m/s, $H-h_o$ is the drawdown at the well in metres and C is a constant taking values between 1500 and 3000. Thus for a drawdown of 10m in an aquifer with a permeability of say 4×10^{-4}m/sec, R would be given to be between 300m and 600m. Unfortunately equation (5) does not reflect the influence of time nor is it sufficiently general to reflect the influence of screen length and depth of aquifer penetration.

Time may be included by considering the well known Theis solution of the equations describing the flow towards wells in fully penetrating confined aquifers, Walton (1970). The Theis equation is,

$$H - h = q/4\pi kH\ W(u) \qquad (6)$$

where

$$W(u) = \int_{o}^{\infty} \frac{1}{u} e^{-u}\, du$$

and

$$u = Sr^2/4kHt.$$

In equation (6), H-h is the drawndown at radius r, q is the yield, k is permeability, H is aquifer depth S is the storage coefficient and t is time. W(u) is a well function which for $u < 0.02$ can be approximated to give.

$$H - h = \frac{q}{2\pi kH} \log_e \left(\frac{2.25kHt}{r^2S} \right)^{\frac{1}{2}} \qquad (7)$$

This can be compared with the equation describing steady state flow towards a similar well having a radius of influence R, Mansur and Kaufman (1962).

$$H - h = \frac{Q}{2\pi kH} \log_e \left(\frac{R}{r} \right) \qquad (8)$$

Comparing equations (7) and (8) gives the following expression for R,

$$R = \left(\frac{2.25kHt}{S} \right)^{\frac{1}{2}} \qquad (9)$$

Thus the somewhat complex equation (6) can be reduced to the more straightforward equations (8) and (9) with equation (9) providing a basis for estimating the distance to the boundary of influence. If for example, $k = 4\times10^{-4}$m/s, H = 20m, S = 0.1 and t = 10 days then equation (9) gives R = 394m, which is of the same order of magnitude as the result of the calculation carried out earlier for Sichardt's equation (5).

Equations (6), (8) and (9) do not, however, suit the majority of cases in that dewatering wells are more often in unconfined aquifers and are partially penetrating with limited screen length. Some guidance on the behaviour of these wells is given by Dagan (1967), but his solutions are limited to small drawdowns although the equations used are linear and may be acceptable for superposition procedures in the case of assemblies of wells. The solutions are also complicated and are tedious to evaluate so it is interesting to note that they may be approximated in a similar manner to the way equations (8) and (9) represent equation (6). The equivalent of equation (8) is

$$H - h = \frac{\alpha Q}{kH} \log_e \frac{R}{r} \qquad (10)$$

and R is evaluated by an equation having the same format as equation (9),

$$\frac{R}{H} = \left(\frac{Akt}{SH} \right)^{1/n} \qquad (11)$$

For a wellscreen length of 0.1H, for example, the relationship between penetration depth and the constants in equations (10) and (11), for t>SH/k, are given in Table I together with the corresponding values for the Theis approximation equation (7).

Penetration depth	n	A	α
0.2H	2.80	2.34	0.224
0.4H	2.03	2.54	0.164
0.6H	1.55	2.01	0.124
0.8H	1.33	1.78	0.106
THEIS	2.00	2.25	0.159

TABLE I

Dagan's solutions are not valid for deep drawdown and may not therefore be valid for use near to the wells which is the region of greatest interest for dewatering.

In addition simple superposition procedures may not be valid. However they demonstrate the way simple equations may be calibrated to provide efficient design tools, offering the promise that one day similar superimposable calibrations will be available for cases of deep drawdown.

For the case of flow towards a slot well as shown in figure 1, it may be shown that

$$L = \left(\frac{3tk}{S} \frac{(H+h_o)^2}{H+2H_o} \right)^{\frac{1}{2}} \tag{12}$$

This equation has the same form as equation (9) indicating that it should be possible to develop equations similar to equation (11) for wellpoint systems. It is interesting to note that equations (9) and (12) imply L>R whereas it is usual in practice to take values for L for design purposes which are an order or magnitude less than R.

Figure 4 illustrates the way in which the need arises to assess the position of the boundary of influence and the consequential drawdown performance of assemblies of partially penetrating wells. It is a section through a typical excavation in East London for the foundation of a pumping station for the storm water and sewage drainage system. Such excavations are usually done within a sheet pile coffer dam which acts as a cut off for the groundwater in the river gravels. The tube wells are used to provide pressure relief in the Thanet sand and there would usually be about six of them for a pumping station. An additional problem illustrated by this example is caused by the presence of sheet piles whose influence on the design of a dewatering scheme is usually very difficult to assess.

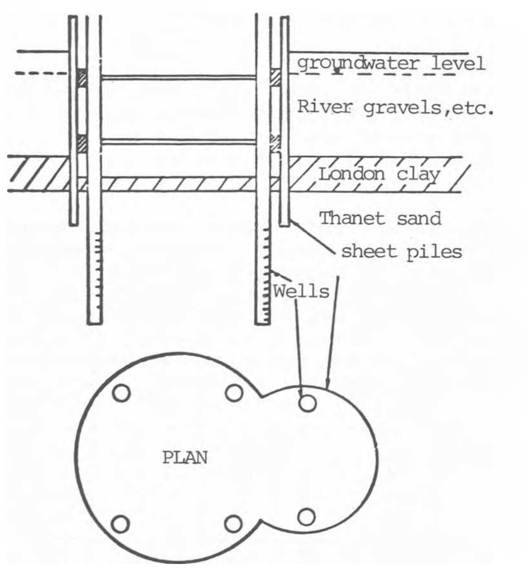

Fig.4 Typical deep well arrangement for pumping station in East London

DRAWDOWN

The primary objective of a dewatering system design is the prediction of drawdown or piezometric surfaces in the neighbourhood of the foundations or, in the case of pressure relief in a confined aquifer, the prediction of pore water pressures. In theory there is no great difficulty in doing this once suitable boundary conditions have been established as simple potential theory can be applied and a solution can be obtained by use of finite difference, finite element or boundary element techniques, Rushton and Tomlinson (1971), Neuman and Witherspoon (1971), Liggett (1977).

However in order to make even these approaches tractable it is usually necessary to convert what is strictly a three-dimensional problem into a two-dimensional problem. Doing this involves making quite serious approximations because the plan of the foundation being dewatered rarely has radial symmetry as illustrated in figures 1 and 4.

Even if a two-dimensional representation of the problem can be made acceptable the numerical methods available require that the problem has to be descretized to a large extent particularly in the neighbourhood of wells, and it is a characteristic of a dewatering system that there will be a lot of these. The extent of the computation involved is in turn proportional to the degree of descretization and therefore numerical solutions tend to be inappropriate for use in day to day design.

The alternative is to use approximate analytical methods which have been calibrated to make them sufficiently accurate by the use of more sophisticated numerical or analytical methods. An example of the calibration of simple equations is given in figure 5. This shows the effect of the presence of sheet piles on flow to a slot in terms of the percentage reduction of flow calculated using the simple Chapman formula as a function of sheet pile penetration. Although no formal complete analytical solutions are available it is possible to calibrate approximate analytical solutions, by using analogue modelling, to provide a reasonably simple and accurate design method for use on a day to day basis.

A common adaptation of deep well theory to the prediction of drawdown near assemblies of wells in unconfined aquifers is unsatisfactory in that it relies on linear superposition in a clearly non-linear situation. A similar calibration of the results of this method by using, for example, a boundary element approach would be reassuring.

PROPERTIES

The final stage in the design of a dewatering system is to assess the relevant properties of the ground in order to arrive at well yields. From equations (3) and (6) it is evident that yields are a function of the permeability of the ground, a parameter whose value can vary in its estimation by more than an order of magnitude. This lack of precision is often

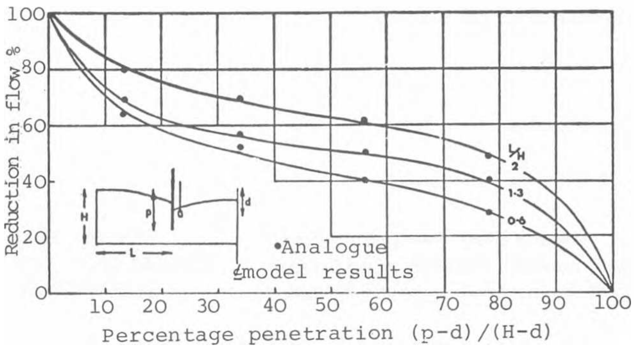

Fig.5 Calibration of Chapman's formula for sheet pile penetration

disconcerting in that is leads directly to a consequential lack of precision in the specification of either the number of pumps required in a wellpoint scheme or in the well capacity, and therefore well diameter, in a deep tube well scheme. The sensitivity of flows to permeability, k, is reduced however if allowance is made for the fact that the boundary of influence will recede quicker as k increases, see equations (9) and (12), which causes a relative reduction in hydraulic gradients. Thus design flows may be viewed as being sensitive to approximately $\sqrt{k}$ rather than k.

One of the most important considerations in the design of a long term dewatering scheme, that is one that is expected to last more than about six months, is groundwater chemistry. Sites particularly sensitive to this factor are those which are near river estuaries. If water is drawn from the estuary it can be found to be partly saline from tidal influences but at the same time well oxygenated from the upstream fresh water supply. This combination is particularly corrosive, Butler and Stroud (1965) In a recent large long term project in the Thames Estuary this situation was compounded by the site being previously occupied by an old acid works and by it having for a neighbour a major scrap metal processor. As a result the rate of erosion on unprotected equipment was extremely rapid with header pipes and pumps only lasting about six months.

CONCLUSION

This paper identifies some of the analytical difficulties which currently face the designer of a dewatering scheme. Difficulties which become particularly acute when the scheme is a large one and the degree of analytical support demanded for a dewatering proposal is extensive.

Many of the problems outlined can be coped with by a site investigation where the Engineer has been able to appreciate the requirements of the dewatering design. If a well designed pumping test has been carried out then this goes a long way to eliminating problems associated with boundary condition specification and determination of relevant soil properties. Appendix I gives the dewatering contractors requirements from a site investigation.

There is still however a volume of research required to provide efficient techniques for the prediction of drawdown levels in the neighbourhood of arrays of dewatering wells. However it is important that the results of this research are presented in a form that is readily accessible to the designer. For example, Dagan's equation, see Appendix II, is too general for dewatering applications and representation of his result in terms of equations such as numbers 10 and 11 are to be prefered.

APPENDIX I

Information needed in a site investigation report by a dewatering contractor.

Description of site location, surface geology and topography.

Location of water sources, old water courses and wells, for supply of jetting water and for dispersal of discharge water.

Location of neighbouring structures for consideration of problems likely to arise through settlement.

Location of services above and below ground which may require diversion, or may constrain location of dewatering equipment.

Description of activities formerly carried out on site, and presently carried out in neighbouring sites.

Boreholes giving full description of geological and soil conditions to the depth of bedding strata if present or, if not, to a depth which is at least twice the proposed excavation depth.

Records of water table depth taken at regular periods.

Results of in situ permeability tests and cone penetration tests.

Results of pumping tests including water levels in observation wells.

Results of laboratory sieve analyses and oedometer tests.

Water analyses giving chemical and physical properties of groundwater.

In addition the dewatering contractor also requires plans and sections of the proposed excavation, the method of excavation and programme and details of any sheet pile cut-offs.

APPENDIX II

Dagan's solution for drawdown, s, and flow, q, towards partially penetrating wells in unconfined aquifers of depth H is

$$\frac{sk\ell}{q} = \frac{1}{2\pi} \left| \ell n \frac{p + \ell/2 + \left|(p + \ell/2)^2 + r^2\right|^{\frac{1}{2}}}{p - \ell/2 + \left|(p - \ell/2)^2 + r^2\right|^{\frac{1}{2}}} \right|$$

$$- \int_0^\infty \frac{\sinh \lambda p/2H \cosh \lambda (1-p/H)}{\pi \lambda \sinh \lambda} \exp\left(- \frac{\lambda kt}{SH} \text{ than } \lambda\right) J_o(\lambda r/H) d\lambda$$

$$+ \int_0^\infty \frac{\exp(-\lambda) \sinh \lambda p/2H \cosh \lambda p/H}{\pi \lambda \sinh \lambda} J_o(\lambda r/H) d\lambda$$

where ℓ is the well screen length, and p is the mid-screen penetration.

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