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The near field boundary of dewatering systems – estimating individual yields for wells operating under gravity flow

by

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ABSTRACT

The subject of this thesis is the estimation of individual well yields, which is a fundamental part of a multi-well abstraction system design. A literature review of the subject shows that the current best practice for estimating individual well yields has several shortcomings and that further research on the topic is required for individual wells operating under gravity flow. The proposal by Sichardt (1927) for estimating the hydraulic gradient at entry into wells is reviewed and his suggestions are compared to the findings in the field. Pumping test data from eight individual abstraction wells, operating under gravity flow in aquifers having a range of permeability values, are presented. The permeability of the aquifer and the implied hydraulic entry gradient into the well were calculated from the data. The findings also show that Sichardt's (1927) formula provides reasonable results for permeabilities in the range 1 x 10-5 up to 2.15 x 10-3m/s. For permeability values below 1 x 10-5 m/s Sichardt (1927) a reasonable estimation, but the results need to be used with caution.

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THE NEAR FIELD BOUNDARY OF DEWATERING SYSTEMS – ESTIMATING INDIVIDUAL YIELDS FOR WELLS OPERATING UNDER GRAVITY FLOW

Christoffel Philippus Botha

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DECLARATION OF AUTHORSHIP

I, Christoffel Botha

declare that this thesis and the work presented in it are my own and has been generated by me as the result of my own original research.

The near field boundary of dewatering systems – estimating individual yields for wells operating under gravity flow

I confirm that:

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3. Where I have consulted the published work of others, this is always clearly attributed;
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6. Where the thesis is based on work done by myself jointly with others, I have made clear exactly what was done by others and what I have contributed myself;
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Notation

A	Area
B	Partial penetration factor of wells
C	Calibration factor
C_{hv}	Coefficient of consolidation for vertical compression of soil under Horizontal drainage
C_v	Coefficient of consolidation of soil
D	Thickness of confined aquifer
	Thickness of compressible layer
d	Depth to water table
	Depth of excavation in cofferdam
	Drainage path length
H	Initial groundwater head
	Excess head in rising and falling head tests
	Applied head in packer test
h	Total hydraulic head
	Groundwater head
	Height of water over weir
h_w	Groundwater head in a pumped well or slot
$(H - h)$	Drawdown
$(H - h_w)$	Drawdown in a pumped well or slot
i	Hydraulic gradient
k	Coefficient of permeability
L_0	Distance of influence for plane flow
l_w	Wetted length of well screen
Q	Flowrate
	Flowrate from a groundwater control system
q	Flowrate from a well
R_0	Radius of influence for radial flow
r	Radial distance from well
	Radius of borehole
r_e	Equivalent radius of groundwater control system
r_w	Radius of well
S	Groundwater storage coefficient
s	Drawdown
s_y	Specific yield
T	Transmissivity
	Time factor
t	Elapsed time
u	Pore water pressure
	Argument of Theis well function
$W(u)$	Theis well function
x	Linear distance

Chapter 1: Introduction

1.1 Background

One of the prime drivers and demands of economic activity is infrastructure. Infrastructure projects involve excavations for tunnels, basements, underpasses, waste treatment systems or deep foundations, which extend underground and below the groundwater level. Excavation and construction below the groundwater table can be difficult and hazardous. The difficulties of excavating below the groundwater level include flooding of the excavation, erosion caused by groundwater seepage, slope instability, ground settlement and contact with contaminates present in the groundwater.

Many of these difficulties can be overcome by managing and controlling the groundwater ingress during construction activities. Groundwater can be controlled by active pumping via wells or sumps, groundwater exclusion methods such as concrete based hydraulic barriers, ground freezing or a combination of these methods (Powers, 1981; Cashman and Preene, 2013; Delleur, 2006; Preene, et. al., 2000). The active pumping techniques used are collectively referred to as dewatering.

Dewatering systems comprise of a group of pumped wells or sumps, which are installed in advance of the excavation. These wells or sumps then interact and subsequently lower the groundwater level. The aim is to lower the groundwater level over a predefined area by continuously pumping from wells so that excavation can take place in workably dry conditions. Once the construction works are sufficiently complete for the structure to be water resistant and capable of resisting hydrostatic uplift forces, the dewatering system can be switched off and removed, allowing groundwater levels to recover. The various types of dewatering system include deep wells, wellpoints, ejector systems and sump pumping (Powers et. al., 1981; Cashman and Preene, 2013; De Leur, et. al., 2007; Preene, et. al., 2016).

Design procedures for dewatering systems fall broadly into three categories; superposition methods which assume that the drawdown of a group of wells is the sum of the cumulative response of the individual wells in the group; equivalent well methods which treat the group of wells as a single well of large diameter; and numerical methods which use computers to model complex boundary conditions (Powers, 1981; Harbaugh, et. al, 2000). A requirement of all the design methods for a dewatering system is to identify the individual well types, sizes, number required and their installation locations (White, 1981).

The theory of groundwater seepage flow towards pumping wells, underpinned by Darcy’s law, is well established and forms the basis for all closed form analytical solutions and modern numerical analysis methods (Darcy, 1856; Dupuit, 1863; Theim, 1906; Theis, 1935). Less well documented are the almost equally significant difficulties associated with establishing the boundary conditions (White, 1981). Two important boundary conditions are the near field at the well, i.e. the achievable abstraction flow from a single well and the far condition, i.e. the distance of influence (Sichardt, 1927; Preene and Powrie, 1993). These boundary conditions are intrinsically interlinked and can be estimated by carrying out in-situ test such as pumping tests, numerical modelling or by using analytical and empirical formulas (Preene, et. al., 2016; Cashman and Preene, 2013; Darcy, 1856; Dupuit, 1863; Theim, 1906; Theis, 1935).

The most commonly used methods for estimating individual well yields are analytical methods based on Darcy’s law (Darcy, 1856). The three main variables in Darcy's law to calculate the flow rates are the area over which flow takes place, the permeability of the porous medium and the hydraulic gradient. The geometry and area over which flow takes place is usually derived by using a simplified hydrogeological concept model, while the importance, potential pitfalls and difficulty in establishing an appropriate value for permeability in an aquifer are well documented, see for example (Preene, et al., 2016; Cashman & Preene, 2013; Preene and Powrie, 1993). One key variable the designer then needs to estimate to apply Darcy's law is the hydraulic entry gradient to the pumping well. Research in the past has attempted to provide guidance on estimating this hydraulic entry gradient (Schultze, 1924; Sichardt, 1927; Roberts, 1988; Arutjunan, 1987; Somerville, 1986; Preene and Powrie, 1993; Preene, 1994; Preene, et. al., 2016). A widely used empirical formula was also presented by Sichardt (1927) for estimating the maximum hydraulic entry gradient to a well.

1.2 Problem statement

In practice, pumping test data are not always available for estimating the parameter values required for designing a dewatering system. This is partly due to cost, and partly because dewatering operations are defined as ‘temporary works’ whereas the purpose of client specified geotechnical investigations is often focused on the design of the permanent works. The design of ‘temporary works’ is the responsibility and preserve of the contractor (FIDIC, 1999). This means that, certainly in the early stages of a scheme development and often at the time of tender, the design, risk assesment and pricing of many dewatering systems has to be undertaken in the absence of site specific pumping test data.

Although numerical modelling tools have become more popular, the cost of modelling is still high compared with analytical and empirical methods (Janssen and Hemker, 2004; Haitjema, 2015). Not only is inverse numerical modelling time consuming and costly, the number of variable parameters result in a "non-unique" solution. For example, a number of permeability and storage coefficient values can give the same answers depending on what other parameters are used (Cashman and Preene, 2013). A further difficulty with the use of numerical models is that, when individual wells are to be investigated, more complex 3D numerical models with special "add on" packages such as MNW2 for MODFLOW are required (Konikow, et. al., 2009). Similarly, various "add on" packages are available for accommodating the far field boundary of a pumping system (Powers, et. al., 2007). The problems are that these complicated models often require site data from pumping tests for calibration and need to be operated by experienced modellers who will understand if the input parameter values used and outputs derived are realistic or not. These problems further increase the cost of modelling. Easy to use empirical and analytical methods are often more cost effective due to additional costs of having trained modellers. The model users are required to have experience in using the software, and also to make judgment on the whether the modelling output is realistic or not. Often site specific data is also required to calibrate the model. Haitjema (2015) describes a case study in which the cost for using simple hand calculations were in the order of $500 versus 2D modelling costing $20,000 and 3D modelling costing $100,000 with no real added benefit in terms of accuracy between the various methods used.

This need for an easy to use cost effective method for estimating individual well yields was recognised in Germany in the 1920s. An empirical formula was developed by Sichardt (1927) for estimating the hydraulic gradient at entry to a well. Sichardt (1927) used data from groups of deepwells operating under gravity flow to investigate this variable. Although this empirical formula has been used extensively in dewatering system design in the past, the methods used in his research have shortcomings related to the calculation of the area over which flow takes place, and the range of permeability values and hydrogeological settings in which it can be applied.

The problem with the calculation of the area over which the flow takes place is related to the assumptions made of the drawdown at the well location. Groups of wells very seldom have groundwater monitoring points in the direct vicinity of the individual wells as opposed to the case of an individual well pumping test. Sichardt (1927) used the general drawdown achieved over the area by the group of wells to estimate the individual flow area at each well. This is not correct as it ignored the presence of well losses and the seepage face created.

Further research carried out on the same subject has placed various restrictions on the range of applicability of this near field boundary formula. (Cashman and Preene, 2013; and Preene, et al., 2016). Preene (1994) suggested Sichardt's formula should not be used at all as it provides over optimistic hydraulic entry gradient values. The restrictions imposed by Preene (1994) and Preene, et. al. (2016) resulted from research carried out on groups of wells, the overwhelming majority operating under vacuum, in low permeability soils. The methods of analysing the hydraulic gradient based on the operation of groups of wells under vacuum have similar difficulties as those Sichardt (1927) faced about the area over which flow takes place and the permeability setting of the dewatering system. Other research has also attempted to provide guidance on the hydraulic entry gradients (Roberts, 1988; Artjunjan, 1987).

A further limitation of previous research is that wells operating under vacuum can mobilise higher hydraulic gradients than wells reliant on gravity flow. Thus, the guidelines derived for the hydraulic entry gradient to wells operating under vacuum are not appropriate to well operation under gravity alone. The current industry best practice for estimating individual well yields is a combination of the research carried out by Sichardt (1927) and Preene (1994). Preene and Powrie (1993) as presented in Preene, et. al., (2016) and does not distinguish between gravity fed wells and wells operating under vacuum.

The need for an easy to use cost effective method for estimating the far field boundary was also recognised by Sichardt (1927). Although his formula is widely used and have formed the basis of dewatering system design from Somerville (1986), the literature presents some shortcomings to this formula related to applicable range of hydrogeological conditions over which it applies and other limitations. (White, 1981; Cashman and Preene, 2013). A significant limitation is that it is a steady state formula; however the distance of influence is theoretically time dependent. Further shortcomings include that the formula is not dimensionally correct and the range of intended use is not clear. An alternative formula was presented by Theis (1935). Although the Theis (1935) formula is an exact mathematical solution, which also incorporates time since pumping commenced, it is based on ideal aquifer conditions, is not straightforward to use and requires storage coefficients values which are not immediately available without pumping test data.

Despite the lack of provenance of the existing industry best practice methods presented in Preene, et. al., (2016) for far distance of influence formulas and the shortcomings of the well yield guidelines identified above, these methods are widely used which. This partly because they are straightforward to apply to potentially complex problems and partly because they provide results that are often found to be useful and applicable.

The problem is that in the literature little research has been carried out on individual well pumping test data over a satisfactory range of permeability settings with the specific objective of producing a methodology for estimating the hydraulic entry gradient to pumped wells.

It is not common for analytical methods to under or over - estimate by three or more times the actual flow rate experienced on a dewatering project (Cashman and Preene, 2013). Overcoming the knowledge gaps identified above will result in the number of individual wells required being estimated more accurately. This will result in the total dewatering system to be designed more coherently with less time lost on site addressing dewatering system shortcomings. Estimating the correct number of wells required in a dewatering system would bring a saving to the client as it limits the installation of too many wells, or the costs associated with re-mobilisation to site for the installation of more wells if too few were installed initially.

1.3 Research objectives

To investigate and overcome the knowledge gaps in previous research, a database of single well pumping tests operating under gravity flow over a wide range of permeabilities is necessary. Data held by a specialist dewatering contractor, WJ Groundwater Ltd, (www.wjgl.com) and an international consultancy, COWI, (www.cowi.com) have been used to generate a set of empirical data.

By using single well pumping test data, the difficulties associated with estimating the hydraulic entry gradient in previous research carried out on groups of wells will be overcome. The area of the well over which groundwater flow takes place can be calculated more accurately as groundwater level monitoring is taking place in the direct vicinity of the pumping well and is not influenced by other pumping wells nearby. The bulk horizontal permeability of the aquifer can also be established more accurately than in previous research on the subject. This data analysis will overcome the difficulties experience by previous researchers related to the area over which flow takes place and the permeability estimation related to groups of pumping wells (Sichardt, 1927; Preene and Powrie, 1993; Preene, 1994). Single well pumping test data will also provide a distance of influence value.

The specific research aims of the project described in this thesis are to:

- Review the literature on the boundary conditions of groups of wells.
- Identify the shortcomings of the current practices and methods used for estimating individual well yields.
- Use original single well pumping test data to calculate the implied hydraulic gradient at entry into wells operating under gravity flow.
- Compare the results obtained against the current practices and guidance for estimating individual well yields.

- Make reasoned recommendations to modify the guidance on the estimation of the near field boundary conditions of wells.

Section 2 provides relevant background information on aquifers, flow to wells, groundwater flow analysis and the near and far field boundary conditions of wells and groups of wells. Section 3 presents the case study data for wells and groups of wells in the form of pumping test and dewatering systems. Section 4 details the analysis of the case study data. Section 5 focuses on the discussion of the data.

Chapter 2: Background

2.1 Aquifer and aquitards

In an aquifer, groundwater is stored in the pore spaces between the soil grains or in between rock fissures and fractures. An aquifer is defined as “the natural zone (geological formation) below the surface that yields water in sufficiently large amounts to be important economically” (Davies and De Wiest, 1966). However, the economical importance of an aquifer is relative, as different industries view the quantity of water supply needed to be economical different e.g. when comparing the water supply required for a town to that required for a single household. In civil engineering, an aquifer yielding even very small quantities of water can result in problems with slope stability for excavations below the groundwater table (Preene, et. al, 2016). In this thesis the definitions of aquifers as presented in the Construction Industry Research and Information Association guidance on groundwater control design and practice will be adopted (Preene, et. al., 2016).

Aquifer: Soil or rock forming a stratum, group of strata, or part of a stratum that is water-bearing (i.e. saturated or permeable).

Aquiclude: Soil or rock forming a stratum, group of strata, or part of a stratum of very low permeability, which acts as a barrier to groundwater flows.

2.1.1 Unconfined aquifers

An unconfined aquifer is an aquifer in which the water table is exposed to the atmosphere through openings in the overlying materials. As there is no confining layer above an unconfined aquifer, groundwater release from the aquifer is replaced by air as illustrated in Figure 1.

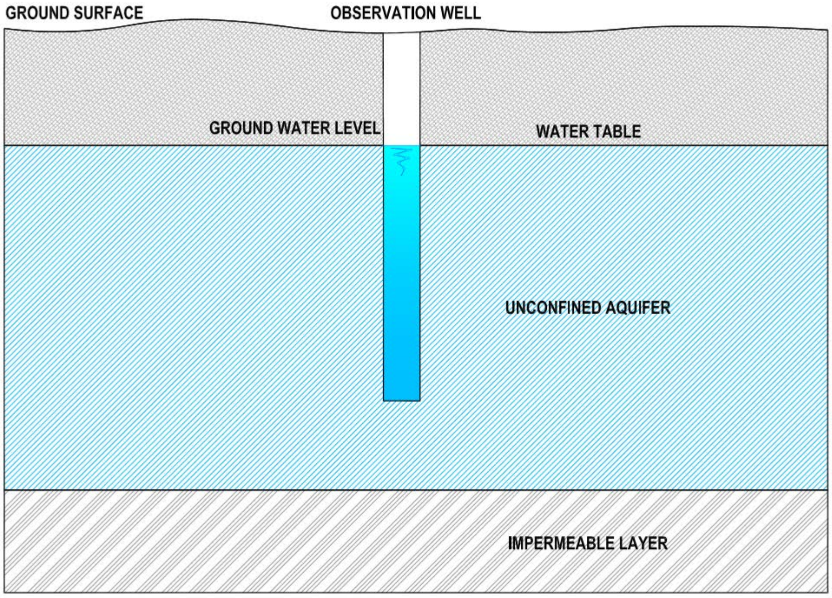


Figure 1: Unconfined Aquifer

When a well is installed and pumped in an unconfined aquifer, a cone of depression is created around the well. As pumping continues this cone of depression deepens and the flow towards the well has vertical flow component in addition to the horizontal flow taking place.

Most construction activities underground take place in the first 20 to 30m below ground level, usually in unconfined aquifer conditions. Most of dewatering projects thus take place in unconfined aquifers.

2.1.2 Confined aquifers

A confined aquifer is an aquifer in which the groundwater is isolated from the atmosphere by an impermeable layer or aquiclude. A confined aquifer is totally saturated from top to bottom and the confined groundwater is generally subject to greater than atmospheric pressures. The pressure distribution is represented by the piezometeric level, the height to which water levels rise if a piezometer is installed into the confined aquifer. A confined aquifer is illustrated in Figure 2.

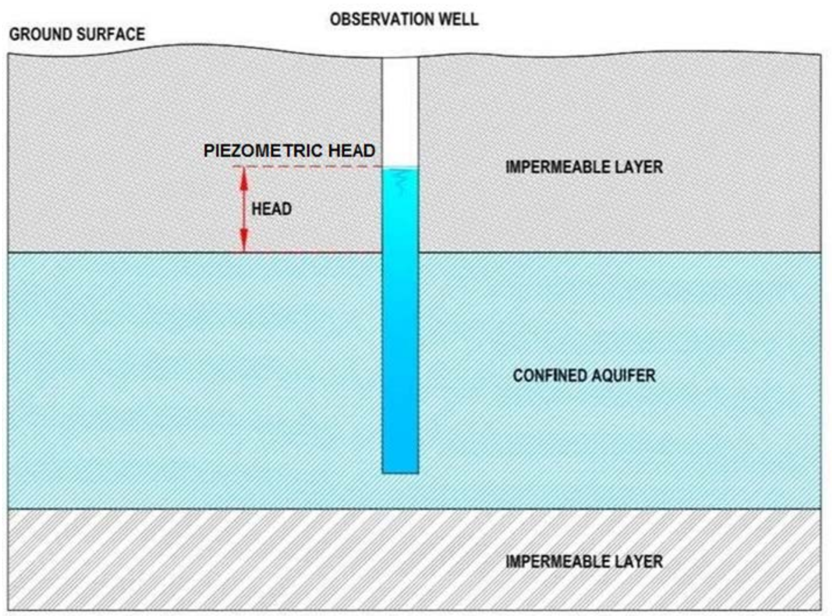


Figure 2: Confined aquifer

When a well is installed that fully penetrates a confined aquifer and pumped, the influence of pumping extends out radially and causes a pressure drop in the aquifer. As long as the water level remains above the top of the aquifer, the flow remains largely horizontal towards the well.

Construction activities taking place in the impermeable parts of a confined aquifer, require little or no dewatering depending on the aquifer parameters of the impermeable layer. Pumping is thus mainly from the confined part of the system to control the pressures acting on the upper impermeable layer(s). Although the flow regimes in confined and unconfined aquifers are different, in both cases there is a head drop in the groundwater at distance away from the pumping well. Thus this study is concerned with both types of aquifers.

2.1.3 Aquifer parameters

Aquifer parameters are used to describe the fixed properties of an aquifer. Site-specific aquifer parameters are usually determined during the site investigation phase of a project. Generally, only a small part of the site investigations carried out during the initial stages of a construction project are focused on the hydrogeological characteristics of the aquifer. This is partly due to cost, but also because early stage site investigations generally focus on the design of the permanent works. Dewatering operations, amongst others, are defined as 'temporary works''. Temporary works design, and additional site investigation required for temporary works design, are usually the responsibility and preserve of the contractor. This means that, certainly in the early stages of a scheme development and often at the time of tender, the design, risk and pricing of many dewatering systems have to be undertaken in the absence of measured values of the key aquifer parameters.

The aquifer parameter values are central to the dewatering system design. The two main parameters defining the hydro-geological properties of an aquifer are the permeability and the storage coefficient (BS 6316:1992). For groundwater abstraction wells, the depth and boundary conditions of the aquifer are also important. The key aquifer parameters are discussed below:

2.1.3.1 Permeability

The permeability of an aquifer is the discharge of groundwater flow through the aquifer under the application of a unit hydraulic gradient. The permeability is also known as the coefficient of permeability or the hydraulic conductivity. In soils the flow takes place in the pore spaces between the individual grains. This is known as the primary permeability. In rock aquifers, flow can take place between the cemented particles but also through fissures and fractures present in the rock mass. The flow through these rock features is known as the secondary permeability. The permeability is an important variable used in dewatering system design. It is directly related to the amount of groundwater that needs to be abstracted by a dewatering system to achieve a given drawdown and is key to the near and far field boundary conditions calculations.

The permeability of an aquifer can be investigated by several methods. These methods can be categorised as either small or large-scale investigations. Small scale investigations are carried out on in-situ boreholes or by the laboratory testing of borehole samples. These methods are usually cost effective and quick to carry out. A disadvantage of small scale permeability testing is that each test determines only the permeability over a relatively small portion of the location from where the sample was taken or the test was carried out. Large scale investigations involve installing one or more pumping well/s and associated monitoring piezometers placed at radial distances from the pumping location. This allows larger portion of the aquifer to be investigated; however these types of tests are relatively expensive and time consuming. The various methods, together with their limitations and shortcomings for estimating aquifer permeability are presented in Table 1.

Location	Test	Comments	Limitations / Shortcoming
In situ – large scale	Well pumping test	Estimates the permeability of a large volume of the aquifer. Added benefit of providing good information on the near and far field boundary conditions	High costs Duration of carrying out a pumping test.
In situ – large scale	Groundwater control trials& dewatering systems	Estimates the permeability of a large volume of the aquifer Added benefit of providing good information on the near and far field boundary conditions	High costs Duration of carrying out a pumping test. Difficult to monitor drawdowns away from the excavation due to permissions and consents for installing monitoring equipment.
In situ – small scale	Borehole test e.g. variable head test, constant head test, packer test	Relatively easy to carry out	Test only a small zone around the borehole Affected greatly by the well installation method Packer tests usually for rock aquifers only
Laboratory	Particle size analysis	Permeability estimated from grading curves by using the method of Hazen. Cost effective method and easy to carry out	Results are dependent on quality of samples obtained Loss of fines or mixing of layered soil can affect results Large quantity of tests to be carried out if a large volume of the aquifer wants to be investigated
Laboratory	Permeameter testing		Results likely to be affected by sample disturbance
Visual assessment			Can give approximate guide to permeability to be used to corroborate results from other test.
Numerical modelling	-	Powerful computing power	Uses groundwater monitoring data to back analyse permeability Aquifer properties still needs to estimated by the operator.

Table 1: Permeability estimation methods and their shortcomings (Preene, et. al., 2000)

Assigning a single permeability value to an aquifer is an idealisation and hence complicated. The complication is related to the intended scale at which the single permeability value is to be used. A permeability value derived for an aquifer over a large area will be different from that derived for a smaller portion of the aquifer. This is due to changing regional topographical features and, over small areas, local inhomogeneities such as secondary features and higher/lower permeability lenses. Over microscopic volumes variation can occur because of changes in soil fabric, as discussed by Rowe (1972). The permeability can also vary depending on the direction in which measured. An example is that if the aquifer is made up of horizontal layers of sand and clay, the bulk permeability in the horizontal direction would be much higher than in the vertical direction. There is thus not one definitive permeability value for an aquifer. In water resources hydrogeology, the areal extent being investigated can stretch over hundreds of km^2. In civil engineering projects, where groundwater control and dewatering are required, an area of up to only a few km^2 may be of interest. While for the design of individual water wells, the area of interest could range from a few hundred m^2 down to only a few m^2. In the case of groundwater control the permeability on a soil fabric scale could be of interest for slope stability in low permeability soils. The permeability range of naturally occurring types of soil varies from less than 10^{-9}m/s for clays to more than 10^{-1} m/s for open gravels. This study is concerned, but not limited, to the range of permeability where wells, sumps or drains for dewatering measures and water supply wells are generally applied. A general range of permeability over which dewatering is carried out is indicated to be in the order of 10^{-7} to 10^{-3} m/s by Preene et. al. (2000). Dewatering is possible at permeabilities outside this range, however the feasibility of groundwater control under these circumstances could be limited by practical or economic factors. In the case of water supply wells, there are usually no practical or economical upper limits to the applicable permeability range as the over pumping of an aquifer by groundwater abstraction is seldom a requirement as with dewatering systems. Figure 3 shows the range of application of pumped wells for groundwater control.

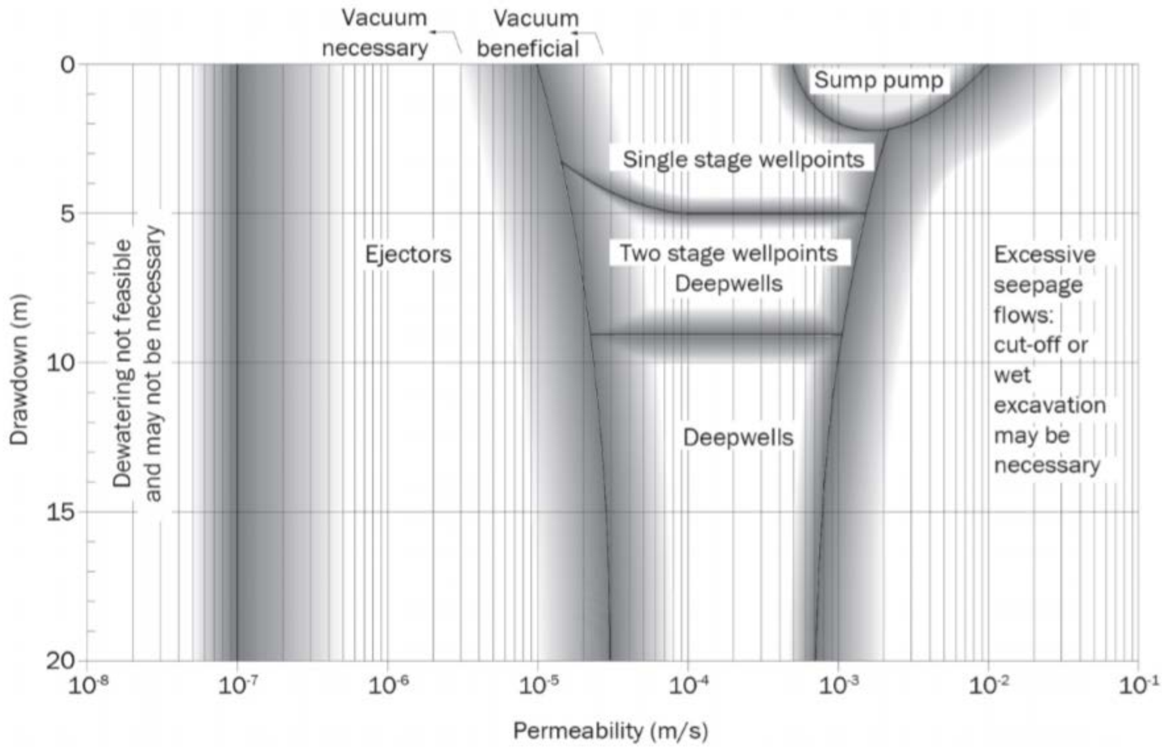


Figure 3: Range of applications of pumped well groundwater control techniques (Preene et al, 2000)

2.1.3.2 Aquifer depth

This aquifer depth is an aquifer boundary condition and represents the bottom of the aquifer. It is important parameter value because it determines the portion of the groundwater that can be abstracted from an aquifer and the amount of drawdown that can be feasibly achieved (White, 1981). The bottom of the aquifer is usually represented by a stratum of low permeability. The aquifer depth can be investigated by reviewing site investigation borehole logs. However, there are a number of problems with establishing the aquifer depth accurately.

Site investigation borehole logs might not be drilled deep enough to locate the low permeability layers representing the bottom of the aquifer. Investigation boreholes are often designed to investigate only the geology a few meters below the planned excavation depth, while the bottom of the aquifer could be well below this level. Even in cases where the site investigation boreholes extend well below the anticipated project excavation depth, the aquifer could still be of such a thickness that the bottom might be deeper still.

Another difficulty is that the permeability of a stratum might decrease with depth. The difficulty is then to estimate when the permeability may be deemed low enough not to yield a significant amount of groundwater that would adversely affect the design flow rates.

When designing dewatering systems, and the bottom of the aquifer is not evident, there is little guidance in the literature of how to overcome this problem. A rule of thumb is presented by Cashman & Preene (2013) that where the aquifer allows, the well depth should at least penetrate one and a half to two time the excavation depth. Another approach might be to carry out a sensitivity analysis by investigating the effect of various aquifer depths on the dewatering system design (Preene, et. al., 2016). The designer also has to make a further judgement as to whether or not the aquifer thickness may be considered to be constant over the project area.

The problem in estimating the aquifer depth manifests itself in this study when analysing single well pumping test data. If the well has not been installed to a known full depth of the aquifer, the bottom of the aquifer was taken as the toe level of the well.

2.1.3.3 Specific yield

The volume of water released per unit drop of the water table per unit horizontal area ($m^3/m^2/m$) is called the specific yield or drainable porosity. S_y. Typical values of S_y are given in Table 2. In finer grained soils with smaller pores, when the water table is lowered, surface tension forces may result in the water not being drained instantaneously and the soil remaining saturated. This process is known as delayed yield.

Aquifer	Specific Yield
Gravel	0.15 - 0.30
Sand and Gravel	0.15 - 0.25
Sand	0.10 - 0.30
Chalk	0.01 - 0.04
Sandstone	0.05 - 0.15
Limestone	0.005-0.05

Table 2: Range of specific yield of different aquifer types (Cashman & Preene, 2013).

The specific yield can be determined by carrying out a pumping test or from case study data in the vicinity if available.

2.1.3.4 Storage coefficient

For confined aquifers the storage coefficient is measured by the amount of water an aquifer can release. A confined aquifer yields water by compression of the aquifer structure causing a reduction in the pore water pressure. The space occupied by groundwater is not replaced by air as in an unconfined aquifer. The volume of water a confined aquifer releases from or takes into storage per unit surface area of the aquifer per unit change in water head is known as the storage coefficient or storativity *S*. The storativity S is dimensionless and for confined aquifers it is in the order of 5 x 10^{-2} to 10^{-5} according to (Marsily, 1986), 10^{-5} to 10^{-3} (Driscoll, 1986) and in the order of 1 x 10^{-3} to 5 x 10^{-4} according to Cashman and Preene (2013). The most accurate method for determining the storage coefficient is by a pumping test.

2.2 Flow to wells

A well can be pumped by a submersible pump installed in the well, by a suction pump from the surface or by means of a nozzle and venturi know as an ejector. A hybrid of these methods can also be used. The water level in the well is then lowered, resulting in a lower pressure within the well than that in the aquifer outside the well. Consequently, since groundwater in the aquifer will move from a point with a high water head to a point of low water head, a flow will be induced towards and into the well.

Groundwater flows towards a well differently in unconfined and confined aquifers. In unconfined conditions, the head of water increases away from the well up to the point of the maximum head where the original groundwater table is unaffected by pumping. It is then apparent that the thickness of the saturated aquifer reduces at the abstraction point as presented in Figure 4. In confined conditions, the low pressure in the well will cause a reduction in the hydrostatic head in the aquifer. The hydrostatic head will be the lowest in the well and increase away from the well up to a point of maximum hydrostatic pressure where it is unaffected by the pumping. The saturated aquifer thickness is generally not reduced during pumping in a confined aquifer.

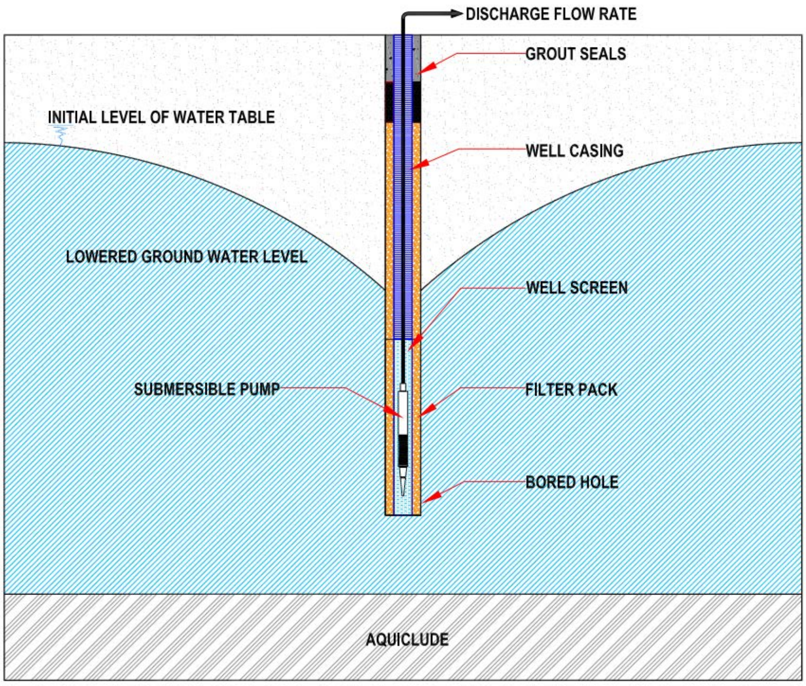


Figure 4 Flow towards a well in unconfined aquifer conditions

The flow pattern towards groups of wells also varies according to the pattern of the well array installed. Individual wells installed in a circle can be considered as one large equivalent well, whereas closely spaced straight lines of wells are modelled as equivalent slots (Powers, 1981). For equivalent wells the flow will converge radially to the well and for equivalent slots the flow converges in a plane manner. For certain groups of well configuration the flow can be radial in some locations and plane in others. Various of well configurations are presented in Figure 4, Figure 5, Figure 6, Figure 7 and Figure 8.

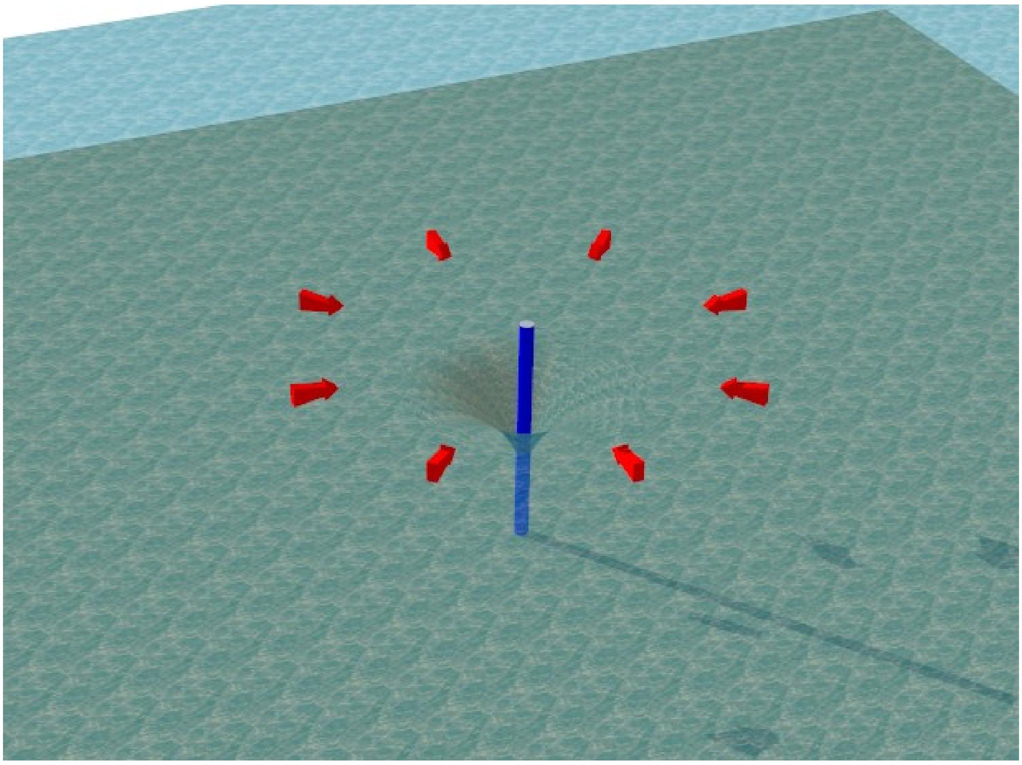


Figure 5: Radial flow to a well

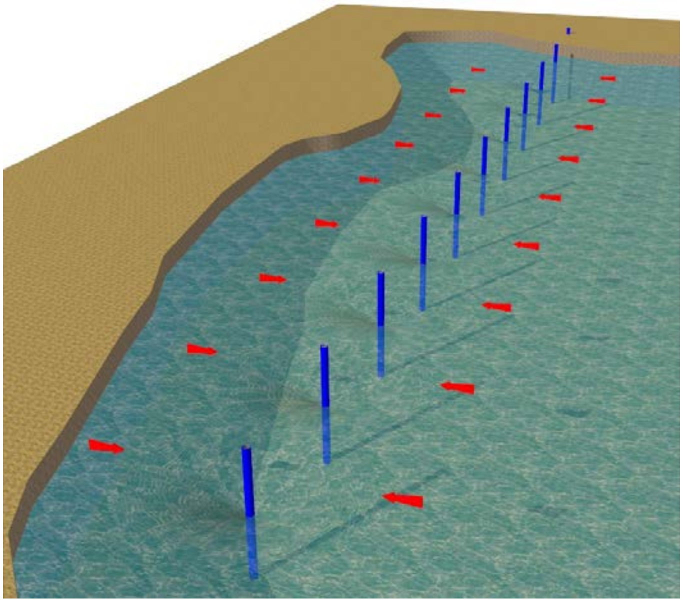


Figure 6: Plane flow to a slot

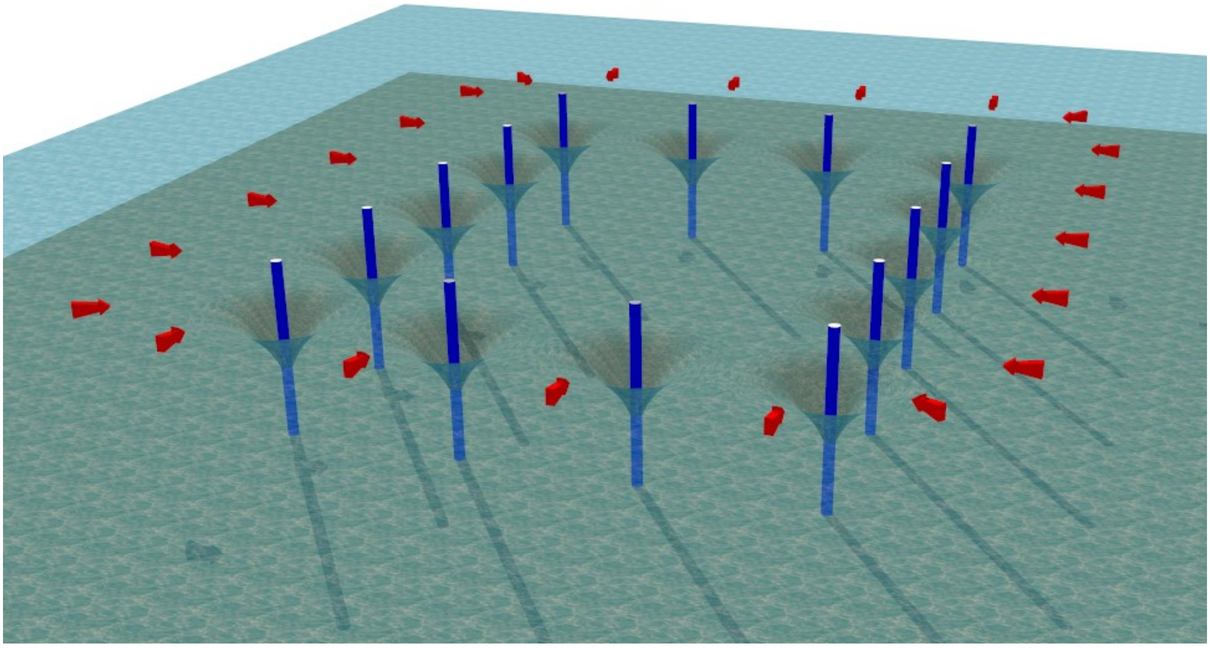


Figure 7: Plane & radial flow to a group of wells

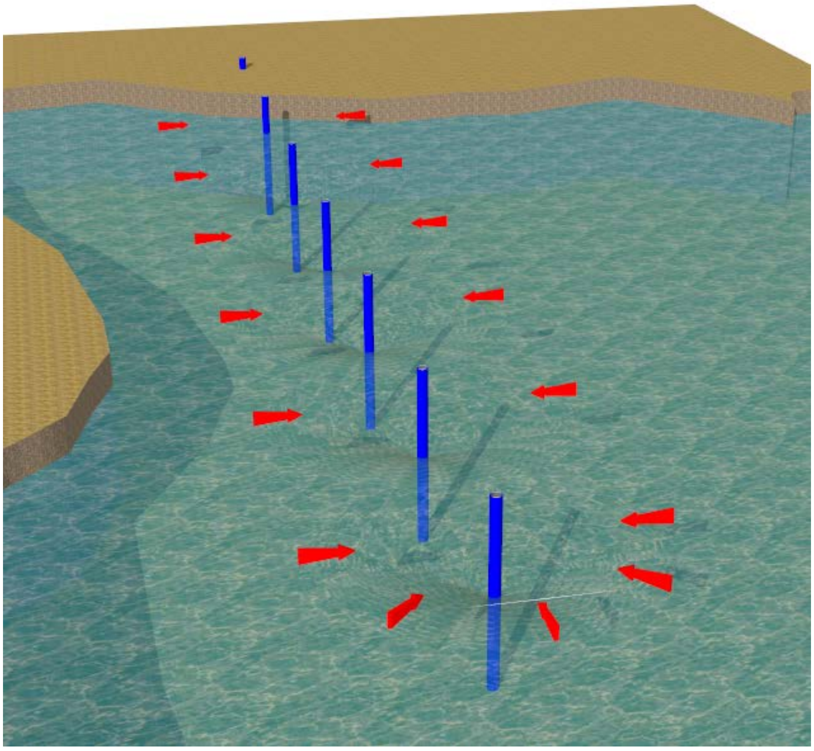


Figure 8: Plane & Radial flow to a line of wells

2.3 Groundwater flow analysis

Methods of varying complexity are available to investigate groundwater flow in aquifers. These methods can be classified into mathematical, empirical and numerical. They can vary from a simple two-dimensional steady state analysis to a transient three-dimensional numerical model back calibrated with actual data from site.

2.3.1 Darcy's Law

In the early 1850's Darcy carried out experiments on groundwater flow in Dijon, France. He published his findings in "Fontaines Publiques de la Ville de Dijon" (Darcy, 1856). In this publication Darcy produced a formula describing a linear relationship between the hydraulic gradient and groundwater flow. Later this became known as Darcy's Law, accepted as the fundamental constitutive relationship in the analysis of groundwater flow (Freeze, 1994).

Darcy's law can be written algebraically as:

$$\boldsymbol{Q = Aki} \tag{1}$$

Where,

Q = Flow rate (m^3/h)

A = Area (m^2)

k = permeability (m/s)

i = hydraulic gradient

In Darcy's Law, Q is the volumetric flow rate; A is the area of porous medium perpendicular to the direction of flow, and i = -*dh*/d*L* is the hydraulic gradient along the flow path, L. h is the head of water. The negative sign implies that the flow is along the direction of decreasing head.

Darcy's Law, being linear, implies that the flow is laminar and not turbulent. Whether flow is laminar and turbulent depends on the Reynolds' number. Following experiments carried out in 1851, Osborne Reynolds introduced a ratio of inertial forces to viscous forces, known as the Reynolds number. Flow is generally considered to be laminar for Reynolds number below 2000 through commercial pipes (Arora, 1989). When the flow becomes turbulent the velocity exceeds a critical value and deviation from Darcy's law may occur.

Darcy's law is also valid for laminar flow through sediments. In porous media, the Reynolds number can be related to the microsocial length of the of the soil particle, *d*.

$$R_e = \frac{\rho u d}{\mu} \tag{2}$$

Where,

ρ = Density

u = Velocity

μ = Viscosity

d = soil particle diameter

In porous media flow, it is assumed that Darcy's law is valid if the Reynolds Number is below a limit of somewhere between 1 and 10, From 10 to 100, the flow is in transition and beyond 100 the state of flow is turbulent inside the pores (Bear, 1979). The effect of turbulent flow is referred to as non-Darcy seepage. Turbulent flow in soils may occur close to the well or sumps in certain unconfined seepage situations.

2.3.2 Dupuit Forcheimer

The French engineer Jules Dupuit formulated a steady state mathematical model of radial flow towards a well by solving Darcy's law for the appropriate boundary conditions (Dupuit, 1863). He visualised a single fully penetrating well placed in the centre of a circular island in an unconfined aquifer. He used the water level changes in the well and the changes in the distance of influence as the boundary conditions to integrate the Darcy formula. The same approach may be applied to plane flow towards a slot.

Thus for plane flow conditions:

$$Q = \frac{kx(H^2 - h_0^2)}{2L} \tag{3}$$

Where,

x = length (m)

H = aquifer thickness (m)

h_o = residual aquifer depth (m)

L = distance of influence (m)

Similarly an equation for radial flow conditions can be presented.

$$Q = \frac{\pi\,\mathrm{k}(H^2 - h_0^2)}{\ln(\frac{R}{r_0})} \tag{4}$$

Where:

R = radius of influence

r_o = equivalent well radius

In order to simplify the mathematics, Dupuit made a number of assumptions:

- the flow is horizontal
- the hydraulic gradient is identical at all points in a vertical cross section of the aquifer
- the aquifer is rigid and the water incompressible
- the aquifer is homogeneous, isotropic and two-dimensional
- the aquifer lies on a horizontal impermeable stratum and extends laterally to infinity
- the pores are completely and instantaneously drained on the passage of the water table
- the flow in the aquifer obeys Darcy's law
- the flow does not vary with time.

These assumptions are rarely the case in reality (Delleur, et. al., 2006).

2.3.3 Theim

The Theim solution applies to confined aquifers and is expressed as (Theim, 1906):

$$Q = \frac{2\pi k D(\mathrm{H} - h_{\mathrm{w}})}{\ln(\frac{R_0}{r_e})} \tag{5}$$

Where,

D = aquifer thickness (m)

2.3.4 Theis

The work done by Dupuit and Theim were for steady state flow conditions. Often it is necessary to evaluate flow conditions at different points in time since pumping commenced. (Theis, 1935) was the first to develop a solution for non-steady state flow. This solution introduces time and aquifer storativity.

Theis's solution can be written as:

$$s = \frac{Q}{4\pi kD} W(u) \tag{6}$$

Where (*u)* is a dimensionless time parameter and $W(u)$ is the well function called an exponential integral and D is the aquifer thickness in m.

$$u = \frac{r^2 S}{4Tt} \tag{7}$$

Where:

t = time (s)

T = Transmissivity

A well function like W(u) and its argument u are also indicated as 'dimensionless drawdown' and 'dimensionless time', respectively. Kruseman and De Ridder (1994) give well functions graphs for several cases. These well function graphs can be used together with observed site data to derive values for u which can then be substituted into the Theis equation. These solutions are based on several simplifying assumptions. Depending on the type of aquifer model being evaluated, certain assumptions upon which the Theis solution is built can be relaxed as described in Kruseman and De Ridder (1994).

2.4 The boundary conditions of pumped wells and groups of pumped wells

Three boundary conditions govern the volumetric extent to which the pumping system will influence the aquifer. These interlinked boundary conditions are; the depth of the aquifer; the amount of water that can be abstracted from a well; and the distance from the pumping well where the groundwater drawdown in the aquifer is not affected (Sichardt, 1927), (White 1981).

2.4.1 Depth of influence

The depth to which a well can be installed to abstract groundwater from the aquifer is governed by this boundary. If a well does not fully penetrate an aquifer, the well is deemed to be a partly penetrating well. With everything else being equal, partly penetrating well will yield less water than a fully penetrating well since there is less aquifer within reach to pump from. Furthermore, a fully penetrating well pumped at the maximum abstraction capacity will also influence the aquifer at a greater distance away from this well, than a partly penetrating well, pumped at maximum capacity under the same aquifer conditions.

2.5 Distance of influence

The groundwater level in and around a well is lowered when water is abstracted from the well. The point at radial distance away from a well, at which there is no lowering of the groundwater in an aquifer is known as the distance of influence. As the time since pumping commenced increases, this distance increases until it is met by a source of recharge e.g. a sea, lakes, open bodies of water, reaches equilibrium with recharge from surface rain or significantly more permeable layers in an aquifer. In the absence of a source of recharge within the affected portion of the aquifer, the aquifer will release stored groundwater in accordance with the aquifer parameters. The distance of influence will then extend out until the maximum rate of abstraction is matched by the rate at which the aquifer can supply water to the well. A steady state condition is then reached. For a well of a given size, depth and drawdown in the well, the closer the distance of influence the higher the abstraction rate will be.

The distance of influence, together with the lowering of groundwater caused by pumping, results in the formation of a cone of depression around the well. The cone of depression has the deepest point at the well location and differs in size and shape depending upon the pumping rate, pumping duration, aquifer characteristics, slope of the water table and recharge within the cone of depression of the well. Figure 9 shows the cone of depression for a pumped well.

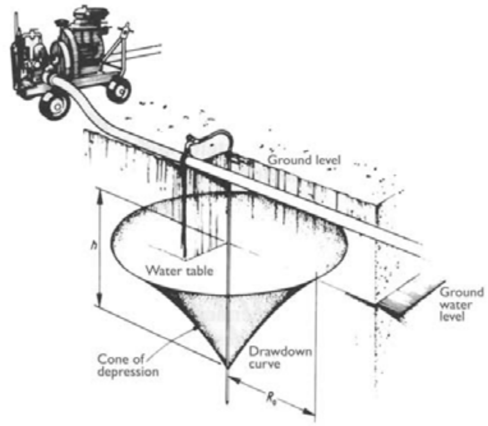


Figure 9: Cone of depression (Somerville, 1986)

When closely spaced wells are pumped from, the cone of depression created by each well extends out until they interact with each other. This principle is exploited in dewatering systems where wells are placed at close proximity around excavations where the lowering of the groundwater is required. Figure 10 shows the cone of depression when two closely spaced wells are pumped. The result of this interaction is shown in Figure 11. From Figure 10 and Figure 11 it may be noted that the drawdown between the wells will be less than the drawdown achieved at the location of the well.

The distance of influence condition can be effectively determined by a pumping tests. In the absence of pumping tests a number of methods are presented in the literature to determine this boundary condition (Sichardt, 1927; Theis, 1935; Kruseman and De Ridder, 1994; Harbaugh et. at, 2000).

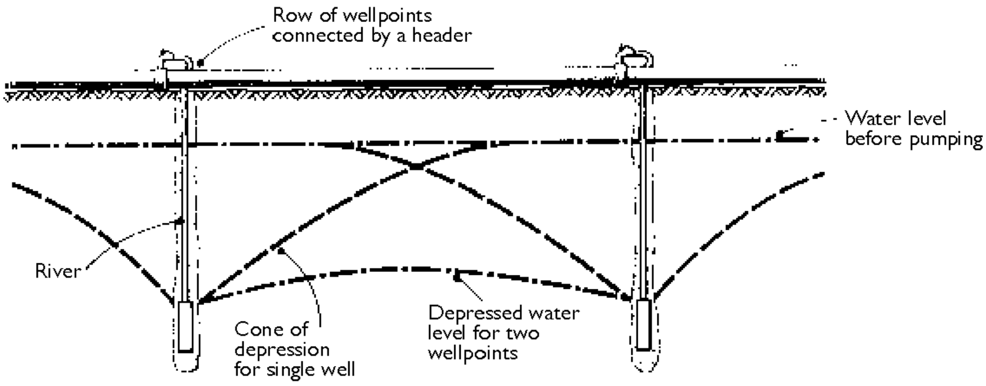


Figure 10: Interaction of cone of depression (Somerville, 1986)

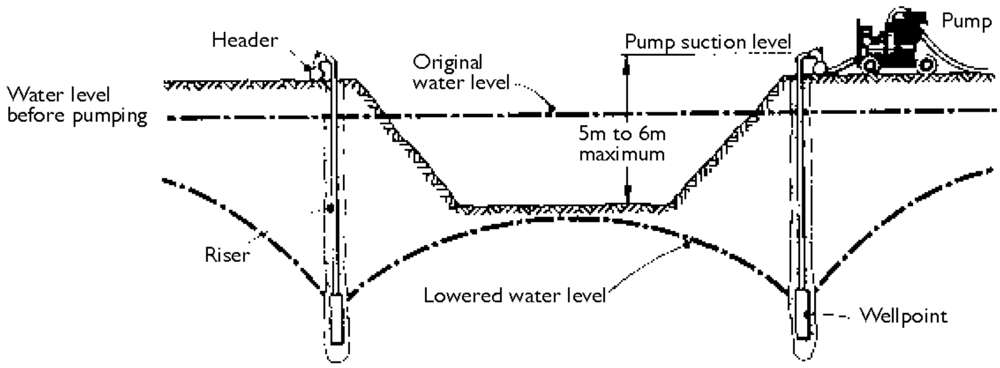


Figure 11: Result of cone of depression interaction (Somerville, 1986)

2.5.1 Far field boundary - distance of influence

Formulae derived from Darcy's law require an estimate of the distance of influence. In this case, the distance of influence is a mathematical convenience, where the sum of the recharge from all sources for a given well location acts as a single equivalent source of large capacity, applied to a vertical cylindrical surface at a fixed distance from the centre of pumping. When the Dupuit Forchheimer approach is used, the radius of influence appears in the equation as a logarithmic function for radial flow. A large error in the radius of influence may only lead to a small error in the flow or drawdown. However, when considering two-dimensional flow to a slot, the distance of influence is inversely proportional to the flow, so very significant errors can occur. The distance of influence is also a function of time. Unless a steady state analysis is opted for, it is important to establish the distance of influence over a specific amount of time, to calculate the rate of pumping required to achieve a drawdown in a particular set of ground conditions.

Methods for calculating the distance of influence can be grouped into three types; analytical, empirical and practical.

2.5.2 Determining the distance of influence

2.5.2.1 Steady state distance of influence

A steady state distance of influence for unconfined conditions can be calculated by using Darcy's law (Darcy, 1856) for plane flow conditions.

$$Q = Aki \tag{8}$$

Where i, they hydraulic gradient, is the ratio of the head drop (*h*) and the distance the head drop takes place over (*l*). Thus:

$$Distance\ of\ influence = \frac{hAl}{Q} \tag{9}$$

2.5.2.2 Transient state distance of influence

The Theis equation can be approximated to the same form as the Dupuit-Forcheimmer radial equation. An equation which estimates drawdown during transient states for radial confined conditions is derived (Kaufman and Mansur, 1962; White, 1981). A transient formula for calculating the distance of influence for plane flow to a pumped slot is presented by Powrie and Roberts (1990).

Radial flow:

$$Ro = \sqrt{\frac{2.25kDt}{S}} \tag{10}$$

Plane flow:

$$Lo = \sqrt{\frac{12kDt}{S}} \tag{11}$$

For relatively compressible soils where water is released largely from storage, the amount of groundwater released is small. Powrie and Roberts, (1990) expressed these equation as:

Radial flow: $$\mathrm{Ro} = \sqrt{2.25\mathrm{C_{hw}t}} \tag{12}$$

Plane flow: $$Lo = \sqrt{12C_{hw}t} \tag{13}$$

Where

$$\mathrm{c_{hw}} = \mathrm{kE'_0/y_w} \tag{14}$$

The analytical expression relating R_0 and L_0 to time may be differentiated to obtain the theoretical rate of change the distance of influence with time.

Radial flow:

$$\frac{\partial R}{\partial t} = \frac{\partial}{\partial t}\left(\frac{2.25kHt}{S}\right)^{0.5} = \left(\frac{2.25kH}{S}\right)^{0.5}\frac{\partial}{\partial t}\left(t^{0.5}\right) = \left(\frac{2.25kH}{S}\right)^{0.5}0.5t^{-0.5} = 0.5\left(\frac{2.25kH}{St}\right)^{0.5} \tag{15}$$

Plane flow:

$$\frac{\partial Lo}{\partial t} = \frac{\partial}{\partial t}\left(\frac{12kHt}{S}\right)^{0.5} = \left(\frac{12kH}{S}\right)^{0.5}\frac{\partial}{\partial t}\left(t^{0.5}\right) = \left(\frac{12kH}{S}\right)^{0.5}0.5t^{-0.5} = 0.5\left(\frac{12kH}{St}\right)^{0.5} \tag{16}$$

The graph in Figure 12 presents the rate at which the distance of influence increases against time for the plane and radial formulas presented. If all variables k, H and S remain constant, it may be observed that the rate at which the distance of influence increases reduces over time and a pseudo steady state is reached.

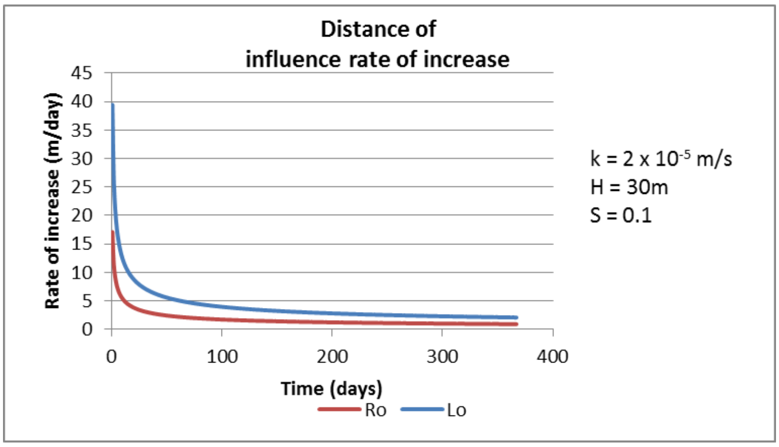


Figure 12: Distance of influence rate of increase

2.5.2.3 Sichardt's formula

An empirical formula is presented by Sichardt (1927).

$$R = C\,(H - h)k^{\frac{1}{2}} \tag{16}$$

Where:

C = constant

This empirical formula provides the distance of influence, R, at steady state conditions and was the basis of a design procedure proposed by Somerville (1986).

Although it gives unrealistic values of the distance of influence at very high and very low permeabilities, it provides reasonable values between these extremities (Roberts, 1988; Cashman and Preene, 2013). Cashman and Preene (2012) notes that this formula was originally developed by Webber (1928). The exact range of applications for which this empirical formula was derived is not clear and not discussed in Sichardt's (1927) publication. A further anomaly with the formula is that the constant would need to have a unit of $\sqrt{\frac{s}{m}}$ for the equation to be dimensionally consistent.

2.5.2.4 Pumping tests

The most accurate way of determining the distance of influence is by means of a pumping test. In a pumping test a large portion of the aquifer is analysed, incorporating both surface and sub-surface boundary conditions of the part of the aquifer affected by pumping. The piezometers placed at radial distances allow mathematical and numerical methods to be applied to this groundwater level data. The Cooper Jacob straight line methods can be applied to estimate the distance of influence from pumping test data. This method is described in (Kruseman and De Ridder (1994) and in section3.2.

A pumping test involves pumping from the aquifer and collecting data on how the aquifer responds. The aquifer is stressed by abstracting water from a well or a number of wells. The data on the aquifer response is then collected by measuring flow and drawdown in the pumping well and monitoring wells placed radially away from the wells.

During a pumping test, a constant discharge test or/and a step test is usually carried out. A constant discharge test involves pumping a well at a constant rate while the effect on the groundwater in surrounding piezometers are measured. The purpose of a constant discharge test is to gather hydro-geological information form a large portion of the aquifer.

During a step test, the well is pumped at a low rate and then in steps the abstraction flow is increased. The flow rate for each step is constant and higher than the previous step. Each step usually last between 30 minutes and two hours (Kruseman and De Ridder, 1992). The purpose of a step test investigates well performance and to determine the rate at which a constant discharge test can be successfully carried out at without the risk of the water level reaching the pump intake level.

Guidance for planning a pumping test is presented in the British Standards (BS6316:1992). The design of the pumping test needs to consider its objectives, the type of aquifer being investigated and the legal requirements of the area the pumping test is being carried out in. In the absence of pumping test data analytical, mathematical and empirical methods are used to investigate groundwater flow.

Item	Single well pumping test	Group of pumped wells
Distance of influence	• Accurately measured by array of piezometers extending away from the pumped well. • Not influence by other closely spaced pumped wells.	• Piezometers at radial distances away from the pumped well are seldom available. • Use of mathematical, numerical or empirical formulas are then used.
Dynamic well level	• Usually accurately measured by pressure transducers.	• Usually measured by less sophisticated water level measuring equipment • Not often measured
Groundwater level at well bore location	• Piezometers closely placed to pumping well location providing good drawdown data close to the well. • Not affected by other nearby pumped wells.	• Piezometer target the drawdown created by a group of wells rather than a specific well. • Piezometers usually not place in the close vicinity of a the wells • Affected by other nearby pumped wells.

Table 3: Advantages of single pumping tests vs. groups of pumped wells for aquifer property analysis

2.5.2.5 Dewatering project case studies

Site data from completed dewatering projects can be used to assess the distance of influence in given ground conditions. Piezometers can be placed at set distances away from the array of pumped wells and used for data collection. A dewatering project often impacts a much larger portion of an aquifer than a pumping test or a dewatering trial. However, there are problems with collecting data at distance from a dewatering project. The main constraint is the placing of piezometers at sufficient radial distance away from the project. Table 4 summarises the associated problems.

Item	Constraint	Result
1	Permission from land owners required.	Increases the cost and time for collecting the data
2	Water levels can be affected by other ongoing dewatering projects.	Data collected not reliable
3	Ongoing and completed projects near the dewatering project.	No access for the installation of piezometers.

Table 4: Far field piezometer placement constraints

The second constraint in Table 4 can cause significant problems if this data are used for future design input values. Unless a study is carried out to investigate the possibility of other dewatering operations influencing the drawdown data collected, it would not be known that the levels are artificially lowered. The data from this method should thus be used with caution.

2.5.3 Far field boundary of influence knowledge gaps

An empirical formula that estimates the distance of influence for a single well was presented by Sichardt (1927). This equation is still widely used today, mainly due to its simplicity. A knowledge gap lies with the shortcomings of this equation, which include the exact origin of the formula, the range of its intended use, the fact that it is not dimensionally correct and that it is not a transient state equation. Further restrictions on the use of this formula are presented in the literature by Roberts, (1988), Cashman and Preene, (2013).

Another knowledge gap is in the utilization of the exact mathematical solutions for transient flow to a well as presented by Theis (1935). The Theis formula and its derivatives as presented by Kaufman and Mansur, (1962); White, (1981) and Powrie and Roberts, (1990) are relatively complicated and a number of key aquifer parameters need to be estimated before such formulae can be used, as discussed in sections 5 and 6 of this thesis. This in conjunction with the ideal aquifer conditions assumed in this method calls into question its applicability to the inhomogeneous and anisotropic conditions in real aquifers. A more serious knowledge gap is that there is no justification in the literature for using either Sichardt (1927) or Theis (1935), both single well formulae, for determining the far field boundary condition for the case of multi-well dewatering systems.

2.6 Maximum Well yield

The maximum well yield is the maximum quantity of water which can be abstracted form a single well. The aquifer parameters and well design is directly related to the maximum well yield. The main components of a typical well are illustrated in Figure 4.

The groundwater level internal to the well, before pumping commences, is defined as the static water level. The water level internal to the well once pumping is commenced, is called the pumped water level or dynamic pumping level. The difference between the original standing groundwater level and the dynamic level is the drawdown. Drawdown also presents the head of water that causes groundwater to flow through an aquifer towards the well and is a function of the rate at which water is being abstracted from the well. The rate at which a given volume of water is extracted per unit of time is termed the well yield. The specific capacity of a well is the yield per unit of drawdown. Figure 13 shows the main components and terminology used for a groundwater abstraction well.

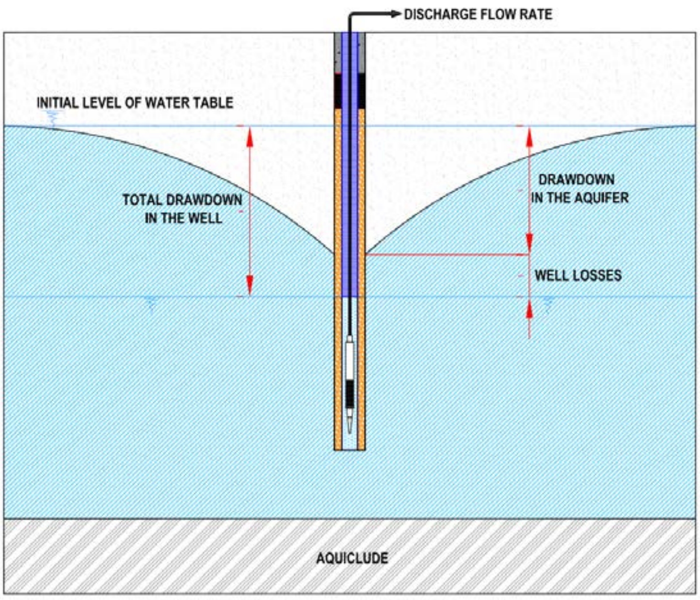


Figure 13: Pumped well terminology

Maximising the well yield is one of the main goals in well design and construction. Characteristics under the control of the designer include the well installation method, drilling fluid used, well bore size, well screen used and filter pack. However, the parameters defining the hydrogeological characteristics of the aquifer are beyond the control of the well designer, even though they may affect the well yield the greatest.

2.6.1 Well installation methods

Wells can be installed by a range of methods and in various sizes. The main methods of well installation are boring, driving, jetting, percussion, direct rotary, reversed rotary and air rotary drilling. More than one drilling method can be used in similar geologies. Particular methods have become dominant in certain areas, because they have the highest penetration rate and are thus cost effective. Further details on which drilling methods are most suitable for particular aquifer strata are given by Driscoll (1986).

The installation method used is influenced by the stratum drilled through and affects the well yield (Preene and Powrie, 1993). Certain drilling techniques can create a “mud cake” or compacted zone adjacent to the well bore, reducing the well yield. On the other hand, other methods could open fissures in rock aquifers, increasing the well yield. The chosen drilling method determined not only by the aquifer parameters, but also according to local experience, engineering judgement and the type of equipment available.

2.6.2 Well bore size

The well bore size affects the well yield. There are two items related to the well bore size; the expected abstraction flow from the well and the possible requirement for a well filter pack.

2.6.2.1 Abstraction flow

Analytical methods based on Darcy’s Law imply that the yield of a well is proportional to the diameter of the well bore, if all other variables are equal (Darcy, 1856). This is because the well will have a larger area of contact with the aquifer and the flow toward the well has to converge less to reach a larger well. However, work carried by Ineson (1959) shows that the relationship is not linear. The case study data from Ineson (1959) are presented in Table 5. Due to this non-linear relationship, rather than increasing the size of the well to increase flow, the size of the well is often determined by more practical factors, e.g. to ensure the proposed size of the submersible pump can be installed into the well or the drilling equipment available (Cashman and Preene, 2013; Driscoll, 1986).

Bored diameter of well through aquifer	203 mm	305 mm	406 mm	457 mm	610 mm
Homogeneous aquifer (intergranular flow)	1.00	1.11	1.21	1.23	1.32
Fissured aquifer	1.00	1.29	1.52	1.61	1.84

Table 5: Case study data of well diameter and normalised well yield (Ineson, 1959)

2.6.2.2 Well filter pack

The choice of well bore size could also be affected by the requirement for a filter pack. When a well is drilled, the borehole is inevitably larger than the well screen. The space between the well screen and bore is called the well annulus. In aquifers with certain properties, a filter pack needs to be installed in the well to avoid constant fines removal from the aquifer while at the same time preserving the well yields. The thickness of the filter pack should be between 0.1 to 0.15m (Preene, et. al. , 2000).

The filter pack has a significant effect on the well yield. If the filter pack is too fine, groundwater will be restricted from entering the well. If the filter pack is too coarse, the continuous removal of fine materials from the aquifer could occur, resulting in damage to the pump and possibly causing settlement of the strata adjacent to the well. Details of efficient filter pack design for wells are presented in Preene, et. al. (2000).

2.6.3 Drilling fluid

The main purpose of a drilling fluid is to support the well bore and remove cuttings from the borehole during drilling. The stability of the strata during drilling and the method of drilling determine whether drilling fluid is needed. Once the well installation is complete, the drilling fluids are flushed out because they are no longer needed and certain drilling fluids may reduce the well yield.

2.6.4 Well casing and screen

A well screen or casing is installed in the borehole once a well has been drilled. A well screen has openings where as well casing does not. The purpose of using a well screen or casing is to prevent or allow water to enter the well from the saturated aquifer portion and to provide a housing for the pump in boreholes with a risk of collapse. The size of the well screen slot is a critical factor in preserving well yield. The important screen criteria and functions are presented by Driscoll (1986):

Criteria:

- Large percentage of open area
- Non-clogging slots
- Resistant to corrosion
- Sufficient column and collapse strength

Function:

- Easily developed
- Minimal encrusting tendency
- Low head loss through screen

The openings in well screens vary in shape and size. They also vary from cost effective hand-made openings made in well casings on site, to expensive highly efficient and long-life models made by machines. The determination of the slot size of the screen is based on the particle size of the aquifer formation. The slot size of the screen should be sufficiently small to retain the well filter pack. The slot size of the well should be approximately the same size as the D_{10} of the filter material (Preene, et. al., 2016).

2.6.5 Well development

After a well has been drilled, the well casing and screen are installed and the filter pack placed, but before the pump is installed, well development takes place. The purpose of well development is to improve the well yield, it includes:

- removal of any residual drilling mud or debris from the filter pack or borehole wall which might otherwise reduce well efficiency
- repair of damage done to the formation by the drilling operation so that the natural hydraulic properties of the aquifer are restored
- increase of the permeability of the aquifer in the immediately vicinity of the well by removing the finer soil particles
- removal of any drilling or development debris from inside the well-liner before installing the pump

A well is developed by applying energy to the filter pack. The aim is to remove an acceptable portion of the fines from the aquifer and the filter pack to increase the hydraulic conductivity between the well screen and the aquifer. There are a wide range of development techniques. For water wells the main methods include over pumping, airlift, surge blocks, jetting and acidisation. The literature also notes the use of explosives in rock wells as a development method (Driscoll, 1986).

Well development usually takes place until the well no longer yields fine materials or the amount of fine materials yielded has reached a suitable limit. Further details on well development methods and fines measurement following well development can be found in the literature (Driscoll, 1986 and Preene et. al., 2000).

Well performance usually declines after some time of operation, which is usually measured in years. Case study data exist where well development was required continuously over a period of months due to well clogging by bio-fouling (Preene, et. al., 2016). When the specific capacity of a well reduces, such that the pumping costs increase or the discharge rate of the well can no longer be maintained, these are indicators the well needs to be rehabilitated. The pumps in the well can then be removed and the well can be re-developed in an attempt to restore the specific capacity of the well.

2.6.6 Aquifer and well drawdown

It is common that the drawdown within the well is more than that directly external to the well bore. This is due to losses in the aquifer, linear and non-linear well losses. The well losses are shown in Figure 14.

2.6.6.1 Aquifer losses

Aquifer losses are the head losses that occur in the aquifer where the flow is laminar. Aquifer losses are time dependent and vary linearly with the well discharge. The drawdown s_1 corresponding to the aquifer losses can be expressed as:

$$s_1 = B_1 Q \tag{17}$$

where B_1 is the linear aquifer loss coefficient. This coefficient can be calculated for confined aquifers using the equation of Theis (1935).

using the Theis equation.:

$$B_1 = \frac{W(u)}{4\pi kD} \tag{18}$$

Where $W(u)$ is an exponential integral:

$$\mathrm{u} = \frac{\mathrm{r}^2\mathrm{S}}{4\mathrm{Tt}} \tag{19}$$

For unconfined aquifers, if the drawdowns are small compared with the initial saturated thickness of the aquifer, the condition of horizontal flow toward the well is approximately satisfied, so that the above equations can be applied. The only changes required are that the storativity *S* is replaced by the specific yield S_y of the unconfined aquifer and that the transmissivity T is defined as the transmissivity of the initial saturated thickness of the aquifer. When the drawdowns in an unconfined aquifer are greater than 5% of the original saturated thickness, these need to be corrected before the equation can be used. Jacob (1947) proposed the following correction:

$$B_c = B_1 - \frac{{B_1}^2}{2D} \tag{20}$$

Where B_c is the corrected drawdown in metres, B_1 is the drawdown in the well and D is the saturated aquifer thickness in metres prior to pumping.

2.6.6.2 Well losses

Well loss can be defined as "The head loss (or additional drawdown inside the well) that occurs when water flows from the aquifer, through the well screen and filter pack into the well itself." (Cashman and Preene, 2013). Well losses are divided into linear and non linear head loss.

Linear well losses are caused by the damaging of the aquifer during drilling and completion of the well. This can be due to compaction of the aquifer material during drilling, plugging of the aquifer with drilling mud, or by head losses in the gravel pack and in the screen. The drawdown s_2 corresponding to the linear well loss can be expressed as:

$$s_2 = B_2 Q \tag{21}$$

where B_2 is the linear well loss coefficient.

Groundwater may flow for some distance over a wide area in a laminar state, but as it approaches a well, the streamlines will converge and the flow velocity will increase to maintain the same volumetric discharge rate. Near the point of discharge, the velocity will often be great enough to be turbulent. Turbulent flow into the well requires more energy and thus a steeper hydraulic gradient than laminar flow. To accomplish this, the water level in the well will have to drop below the level required for laminar flow. Non-linear well losses are head losses that occur in the zone adjacent to the well where the flow is usually also turbulent. The drawdown s3 corresponding to this nonlinear well loss can be expressed as:

$$s_3 = CQ^P \tag{22}$$

where C is the nonlinear well loss coefficient and *P* is an exponent. These losses together are responsible for the drawdown inside the well being much greater than would be expected on a theoretical basis. The general equation describing the drawdown in a pumped well as function of aquifer/well losses and discharge rate is thus:

$$s_w = s_1 + s_2 + s_3 \quad (23)$$

Thus:

$$s_w = (B_1 + B_2)Q + CQ^P \quad (24)$$

Thus:

$$s_w = BQ + CQ^P \quad (25)$$

Jacob (1947) used a constant value of 2 for exponent Lennox (1966) suggested a value of between 1.5 and 3.5 and Delleur, et. al., (2006) noted that this value may be higher for fractured rock. The values of parameters B, C and P can be found from the analysis of the analysis of the step test data gathered during the pumping test. Aquifer losses and well losses associated with the pumping of a well are presented in Figure 14.

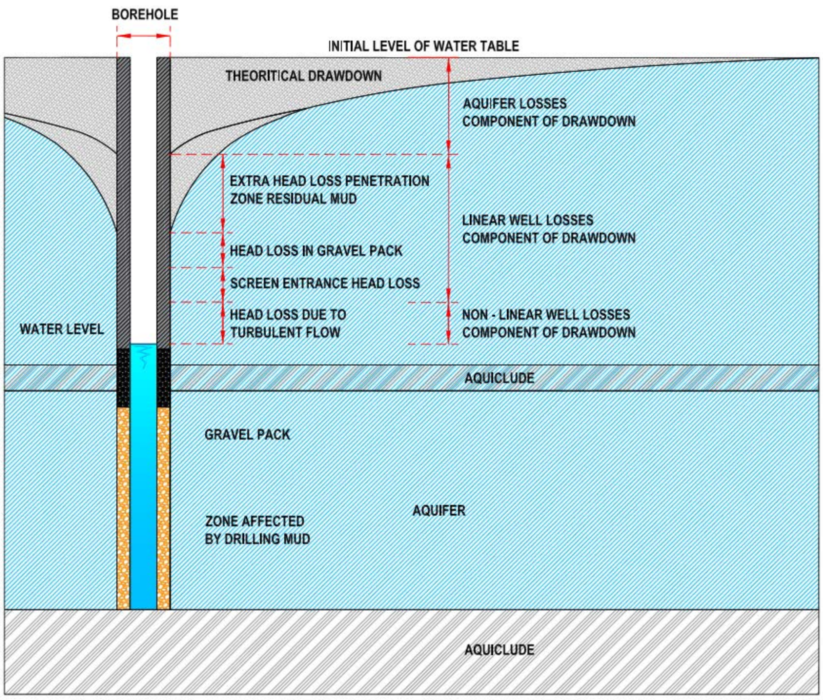


Figure 14: Well losses

The drawdown at the individual well location would thus be lower than at locations between two wells due to the aquifer losses. Figure 9, Figure 10 and Figure 11 illustrate how pumped wells at close spacing result in the higher drawdown between the wells and at the well location.

2.6.7 Determining well yields

Well yields can be determined by carrying out an in-situ test on site, such as a pumping test, analytical, empirical and numerical methods.

2.6.7.1 Pumping tests

The most accurate method to investigate and estimate well yields is to carry out a pumping test (Cashman and Preene, 2013). A step test or constant discharge pumping test will give an indication of how the aquifer responds to pumping and give valuable information on the performance of the well (Kruseman and De Ridder, 1994). All other methods of estimating the expected well yield in the absence of a pumping test will be based on analytical, empirical or possible past experience. However, pumping tests are not often carried out as discussed in Chapter 1.

2.6.7.2 Analytical methods

Analytical methods for calculating well yields are based on Darcy's law. Using Darcy's law (1856) using the area over which the flow takes place, the permeability and an estimation of the hydraulic entry gradient (Sichardt, 1927; Preene and Powrie, 1993; Preene, 1994; Cashman and Preene, 2013).

2.6.7.3 Empirical methods

A widely used formula for estimating the hydraulic entry gradient is presented by Sichardt (1927). He based his research on the assumption made by Kyrieleis (1919) that there is a connection between the permeability and the hydraulic gradient. Sichardt (1927) argued that a maximum hydraulic gradient had to exist as in practice it was not possible to increase the abstraction rate and drawdown internal to a well beyond a certain point. Sichardt (1927) references research carried out by Schultze (1924) to estimate individual well capacity. Schultze (1924) based his research on dewatering system data proved by Prinz (1919) to derive a recommendation for a limit on the hydraulic entry gradient. A hydraulic entry gradient limit between 1.5 to 2.5, which are valid for permeability ranges smaller than 1 x 10^{-4} m/s, was derived by Schultze (1924). The range of permeability values used in Sichardt (1927) research ranged between 1.03 x 10^{-4} m/s up to 5.8 x 10^{-3} m/s for deepwells (Sichardt, 1927). Sichardt (1927) concluded that the limits imposed by Schultze (1924) are only valid for permeability values between 1.5 x 10^{-3} m/s to 4 x 10^{-4} m/s as Schultze (1924) overestimated the amount of wetted screen in his research (Sichardt, 1927).

Building on research carried out by Theim (1906), Schultze (1924) and using site data from Schulttze (1924), Kyrieleis (1919), Sichardt (1927) used data from dewatering systems from the

Netherlands and East Germany to derive a formula for estimating the maximum hydraulic entry gradient to individual wells (Sichardt, 1927). Site data from five dewatering projects were used. The data available included the average individual well yield, the well diameter, the permeability of the soil in which the dewatering system operated and the wetted screen length. Little detail on the geological setting of where the systems operated is mentioned in the literature, except that it was in sands. Using Darcy's Law, Sichardt (1927) calculated the hydraulic gradient for each dewatering case study. The results are presented in Table 6. Sichardt (1927).

Construction site		k-value (m/s)	Well diameter (m)	Wetted screen length (m)	pump rate per well (l/s)	calculated gradient outside well i
watergate in Wemeldinge in Holland [1]	east side	0.000123	0.15	5	1.81	6.25
	north side	0.000197	0.15	5	2.22	4.78
	west side	0.000103	0.15	5	2.15	8.87
Groundwater lowering at central station in Leipzig		0.0053	0.15	5	8.33	0.666
Northern watergate in Plötzensee		0.0014	0.18	4	4.9	1.55
Groundwater lowering in Tegel		0.002	0.15	4.85	5.95	1.3
Shaft I of the Matador mining company in Senftenberg		0.0003	0.15	4.87	3	4.44
		0.0003	0.15	5.82	2.92	3.54
Groundwater lowering in Gartenfeld (Berlin)		0.0028	0.15	5.5	9.44	1.3

Table 6: Sichardt (1927) case study data (Reproduced from Sichardt 1927).

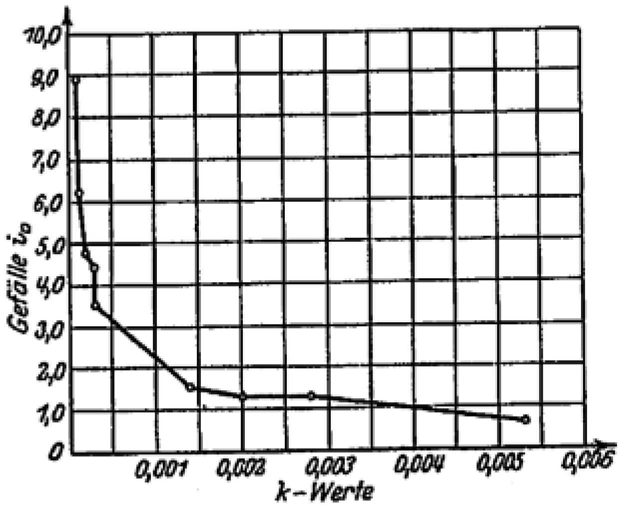


Figure 15: Implied hydraulic gradient versus permeablity. (Sichardt,1927).

Figure 15 presents Sichardt (1927) hydraulic gradient plotted against permeability. The permeability is in m/s. The data plotted used the algebraic expression for a hyperbola and the following coordinates to derive a value for m = 14.9 (m ≈ 15) and n = 0.52 (n ≈ 0.5). Sichardt (1927) carried this out by hand calculations as computers were not available at that time.

$$m.i = \frac{1}{k^n} \tag{26}$$

Substituting for m and n:

$$i_{max} = \frac{1}{15\sqrt{k}} \tag{27}$$

Several researchers suggest the use of Sichardt's formula for determining the hydraulic gradient (Powers, 1981), (Hausman, 1990). Preene (1994) notes that Sichardts formula should not be used as it provides over-optimistic values for the hydraulic gradient at entry to the well, however does not state a reason/s for this observation.

Another method for estimating individual well yields is provided by Somerville (1986) in the form of data sheets from which the maximum well yields for vacuum wellpoints, wells and ejectors can be estimated if the permeability and well diameter are known. However, the research these datasheets are based on is not mentioned explicitly in this publication (Somerville, 1986).

Research by Preene and Powrie (1993) and Preene (1994) used data from dewatering projects to provide guidelines for the hydraulic gradient to be used when calculating well yields. Preene and

Powrie (1993) concluded that the hydraulic gradient for ejector wells should be between 2 and 4, and for vacuum wellpoints between 4 and 10. Preene (1994) indicates that the hydraulic gradient for an ejector system should not exceed 10, and for a vacuum wellpoint system should not exceed 4. No recommendations for the hydraulic gradient to be used for deepwell systems were made. The permeabilities over which the research was carried out ranged between 1.5 x 10^{-6} m/s and 1 x 10^{-4} m/s for ejector dewatering systems and 1 x 10^{-5} m/s to 4.5 x 10^{-5} m/s for vacuum wellpoints. One deepwell dewatering system data point was used in a soil with a permeability in the order of 4 x 10^{-5} m/s. Further restrictions have been placed on the research carried out on this topic prior to 2000 by Preene et al. (2016). Preene et. al. (2016) suggest that the Sichardt formula should only be used for permeability values greater than 1 X 10^{-4} m/s for all types of wells, and a value of 6 used for permeability values lower than 1 X 10^{-4} m/s. The guidelines provided by Preene and Powrie (1993) and Preene (1994) for estimating the hydraulic entry gradient to a well are presented in Figure 16 and guidelines provided by Preene, et. al., (2016), are presented in Table 7 and Figure 17.

Type of well	Preene 1994	Preene and Powrie 1993	(Preene et al, 1990)	Sichardt 1927
Wellpoint	up to 4	between 2 and 4	k < $1x10^{-4}$ m/s → i = 6 k > $1x10^{-4}$ m/s → i_{max}	$i_{max} = \frac{1}{15\sqrt{k}}$
Ejector	up to 10	Between 4 and 10	k < $1x10^{-4}$ m/s →i = 6 k > $1x10^{-4}$ m/s →i_{max}	$i_{max} = \frac{1}{15\sqrt{k}}$
Deepwell	n/a	n/a	k < $1x10^{-4}$ m/s → i = 6 k > $1x10^{-4}$ m/s →i_{max}	$i_{max} = \frac{1}{15\sqrt{k}}$

Table 7: Current guidelines for estimation of the hydraulic gradient

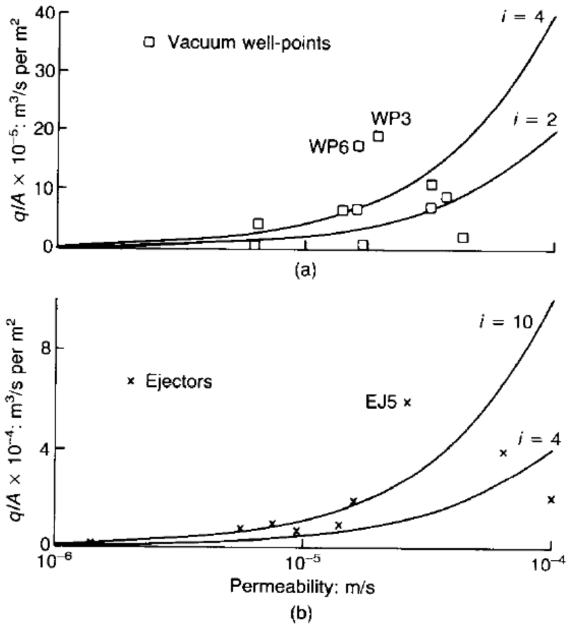


Figure 16: Inferred hydraulic gradients at entry to the well: (a) vacuum well (b) ejector wells based on the permeability and wetted screen area of groups of wells under vacuum flow (Preene and Powrie, 1993).

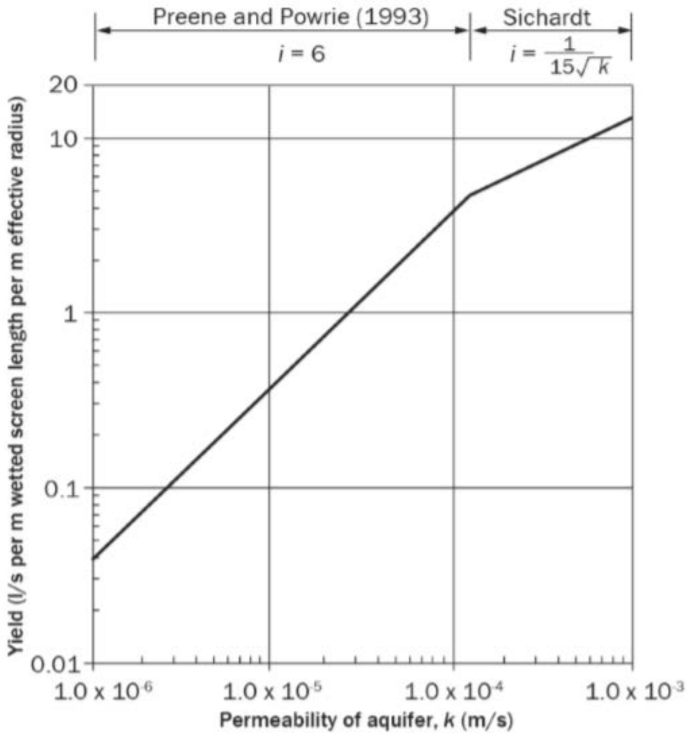


Figure 17: Preene, et. al., (2016) guidelines on estimating well yields

2.6.7.4 Numerical methods

The abstraction rate for individual wells can be analysed using numerical modelling. Numerical modelling provides a powerful tool for analysing complex groundwater flow situations (Harbaugh, et. al., 2000). Until recent, numerical models had limited capacity to handle the detailed flow conditions in and around a well. Past approaches assumed that the modelled well is connected to a single node of the grid and the water level in the well is identical to the head at the connected node (Konikow, et. al, 2009). These methods did not allow for the numerical model to distinguish between the pumped water level in the well, the groundwater level at the well bore location and the well losses taking place as discussed in section 2.6 of this document and presented in Figure 14. Recently multi node well package (MNW2) for MODFLOW was developed which allows flow to be calculated into a borehole through a seepage face and the option to specify the pump intake at any level in the well (Konikow, et. al, 2009). This package thus incorporates the head losses associated with flow to a well including linear and non-linear head losses. There is also an option not to include non-linear head losses in the well analysis.

There are shortcomings of using numerical modelling to investigate well yields in the form of the cost of modelling, the experience of the user and the input data required. The cost of modelling and the experience requirements of the user have been discussed in section 1 of this document. A more serious shortcoming of this method is the detailed input required to calibrate a pumping well using MNW2. Input is required such as the Kskin , which is the gravel pack or formation damage value causing the seepage face. An abstraction rate for the well also needs to be chosen or the drawdown in the well inversely estimated. Without pumping test data, estimating these values is difficult.

2.6.7.5 Well yield knowledge gaps

Research by Sichardt (1927), Preene and Powrie (1993) and Preene (1994) was based on data from dewatering systems consisting of pumped groups of wells. Darcy's law was used by these researchers to calculate the hydraulic gradient. This required the estimation of the abstraction rate per well, the area from which water is abstracted from the aquifer, and the permeability.

The flow per well was calculated by taking the average total flow of the system and dividing it by the number of wells pumping. It was thus assumed that every well contributed an equal amount of flow. However Preene and Powrie (1993) and Preene (1994) note that individual well flow rates on site may vary by a factor of 100.

The area available from which water was abstracted by the aquifer was calculated by taking the average drawdown, the well bore diameter and the depth of the well. As discussed in section 2.5

the drawdown at each individual well will be greater than the drawdown between the wells. The methods used by Sichardt (1927), Preene and Powrie (1993) and Preene (1994) would thus overestimate the wetted screen area. This shortcoming was also identified by Sichardt (1927).

The permeability values were determined from in-situ borehole tests, laboratory tests and back calculation from dewatering projects for the research carried out by Preene and Powrie (1993) and Preene (1994). How the permeability values were derived by the data used in Sichardt (1927) is not discussed in his research. To back calculate the permeability from the dewatering projects, the distance of influence needs to be estimated. This was done by using the formula presented by Sichardt (1927) for estimating the distance of influence. The shortcomings of this formula have been discussed in section 2.5. The shortcomings of using in-situ borehole tests and laboratory permeability tests and applying them to a larger portion of the aquifer have been discussed in section 2.1.

The current best practice guidelines for estimating well yields are based on the work carried out by Sichardt (1927), Preene and Powrie (1993) and Preene (1994) and presented in Preene et. al., (2000). These methods would thus incorporate the shortcomings on the individual well yield estimation, distance of influence estimations, permeability calculations and drawdown at the individual wells calculated by previous research discussed in this chapter.

Furthermore, the majority of the research carried out by Preene (1994) and Preene and Powrie (1993), was for groups of wells operating under vacuum conditions. As discussed, wells operating under vacuum conditions can create larger entry gradients than wells operating under gravity fed conditions.

It is also feasible that the recent advances made over the past decades with regard to drilling and well installation technology could result in more efficient wells being utilised.

In conclusion, there is a clear requirement to review these guidelines and provide more coherent and defined methods for establishing well yields when pumping test data are not available for the use in well design input. The shortcomings of using dewatering system data, which consist of groups of wells, have been discussed in this chapter. Figure 18 presents the previous research carried out and the knowledge gaps identified together the respective ranges of permeability values. This study proposes to address these knowledge gaps by analysing datasets from pumping test data on gravity fed deep-wells.

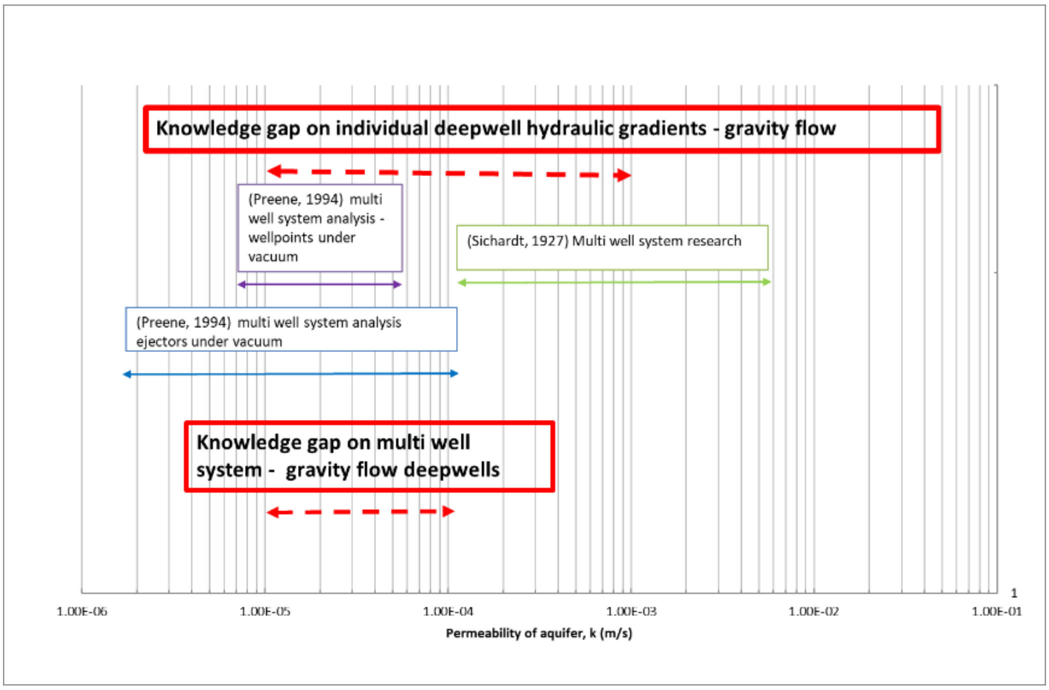


Figure 18: Previous research on hydraulic entry gradients into wells and the identified knowledge gaps

Chapter 3: Site data

In this chapter previous unpublished field data from single well pumping tests, operating under gravity flow conditions, are presented. The data sets were provided by a specialist dewatering contractor, WJ Groundwater Limited (www.wjgl.com) and a Civil Engineering Consultancy, COWI (www.cowi.com).

3.1 Single well case study data

The purpose of the pumping tests in these data sets were to derive key aquifer hydraulic parameter values with the intention of designing multi well dewatering systems. There are no "typical" pumping test designs or aquifer conditions in which a pumping test can be carried out (BS6316:1992). However, pumping tests that intend to provide hydraulic parameters for the design of dewatering systems, are usually carried out over permeability ranges where dewatering is deemed to be a feasible solution for groundwater lowering. This range is typically between 1 x 10^{-7} and 1 x 10^{-3} m/s (Roberts and Preene, 1993). For systems under gravity flow conditions, a permeability range of 1 x 10^{-5} up to 1 x 10^{-3} m/s is recommended. The groundwater levels and abstraction flow rates gathered during the pumping tests can then be analysed to derive the key aquifer parameters used in the multi well dewatering system design.

This chapter discusses the criteria on which the selection of pumping test data for this study was based. It should be noted that to overcome the identified knowledge gaps, pumping test data over the range of permeabilities where dewatering usually takes place, are required. Thus, the selected pumping test data were analysed and the aquifer permeabilities derived to review the suitability of the test data.

3.1.1 Pumping test type

A pumping test is one of a variety of types of tests that are available, including equipment tests, constant discharge tests and recovery tests. Each of these tests serve a specific purpose and the results are analysed using different methods (BS6316:1992). However, not all these tests are always carried out during a pumping test as the overall purpose of the pumping test needs to be considered. For example, a step test would provide information on well efficiency, but the constant discharge test is deemed more reliable for deriving key aquifer parameters (Kruseman and De Ridder, 1994).

To overcome the knowledge gaps identified in this study, the constant discharge test is of most interest as it will provide data on the individual well yield, bulk horizontal permeability of the aquifer, storage coefficient and distance of influence. This type of test often forms the basis for multi well dewatering system design (Cashman and Preene, 2012).

3.1.2 Pumping well

For ease of analysis select pumping tests where the pumping well fully penetrates the aquifer being investigated were selected. Tests where it is known that the well is only partly penetrating, have been omitted from this study. In cases where the bottom of the aquifer is not immediately evident, it has been assumed at first instance that the well fully penetrates the aquifer.

Furthermore, only pumping tests where the pumping wells and associated piezometers are screened in the same strata have been used in this study. Tests where a pumping well penetrates multiple aquifers, but the piezometers are only screened in set portions of the aquifers, and vice versa, have been omitted. Similarly, tests where the pumping well penetrates multiple aquifers and the associated piezometers also penetrate multiple aquifers have not been used. Pumping tests where the piezometer and the pumping well are screened in the same stratum will give information on the bulk horizontal permeability of that stratum and the associated pumping rate, as opposed to the average bulk horizontal permeability and abstraction rate of all the various strata screened (Kruseman and De Ridder, 1994), (Cashman and Preene, 2013). No limiting criteria were imposed on the drilling and installation methods of the pumping wells, other than the fact that the well had to have been developed before use.

3.1.3 Piezometers

The groundwater level data from the piezometers are important to determine the key aquifer parameters of a large part of the aquifer (Kruseman and De Ridder, 1994). Pumping tests with a minimum of three piezometers were considered. No limiting criteria were imposed on the drilling and installation method of the piezometers other than that the well have been developed before use. The frequency of level measurement of the pumping tests were carried out as per the British Standards (BS6316:1992).

3.1.4 Flow measurement

The abstraction rate obtained during the pumping test is important as it is used in the analysis to derive the key aquifer parameters. Pumping tests where the abstracted groundwater was discharged via a V-notch discharge tank were considered. This type of tank contains a weir of known size and shape such that the head above the notch of the V is related to the flowrate over this weir.

3.1.5 Pumping test data

Data gathered for 7 No. tests are presented and analysed in this section. The purpose of the analysis was to calculate the bulk horizontal permeability to ensure that data covering a range of permeabilities were selected.

This chapter describes the single well pumping test data, the data analysis methods used on the single well site data and presents the multi well system data. The methods used for single well pumping test and dewatering system analysis were based on methods available in the literature (Kruseman and De Ridder, 1994; BS6316:1992; Cashman, Preene, 2013; and Preene, et. al., 2016). No new analysis methods are used beyond already established methods.

3.2 Pumping test data and data analysis

3.2.1 North Woolwich pumping test (Woolwich, 2010)

In July 2010 WJ Groundwater Limited were commissioned to carry out a pumping tests at the site of the proposed North Woolwich Portal of the Crossrail Thames Tunnel in the United Kingdom. The main contract works involved the construction of a TBM reception chamber and tunnel approach structure comprising cut and cover and retained cut sections. One aim was to derive key aquifer parameters to be used for the design of a dewatering system for the superficial aquifer. The gravel aquifer was present from +96.02 mD to +90.52 mD giving an aquifer thickness of 5.5m. The piezometric head before the test was started was at +99.91 mD. Note that on Crossrail projects the reduced level refers to an adjusted “Tunnel Datum level”. This is the case on this case study as well. The bottom of the aquifer was represented by a chalk layer. It is feasible that the chalk layer could contribute an element of vertical flow to the upper gravel layers. For analysis, it has been assumed that the chalk layer is the bottom of the aquifer and that the well is fully penetrating. This assumption is reviewed further in section 4 of the study. The flow regime for the gravel on this project is expected to be matrix flow. One fully penetrating deep well and eight piezometers screened in the same stratum were used to carry out the test. The pumping test commenced on 14 Sept 2010 with water being pumped to waste at approximately 20.1 l/s. A failure of the duty generator occurred at approximately 21:00 on 16 Sept 2010, which caused the standby generator to start immediately (within 1 minute) allowing the test to continue. There was a tidal influence on the groundwater levels due to a river near the project. The drawdown after approximately three days of pumping to waste has been calculated using tidal peak water levels before the start of pumping and before the pump was switched off to compensate for the tidal influence.

To investigate whether this pumping test meets the criteria set in section 3.1 of this document, analysis of the pumping test data was carried out. Pumping test data can be analysed in many ways as summarized by Kruseman and De Ridder (1994). The different methods are based on different aquifer assumptions. The most commonly used methods fall into two main categories. Curve fitting methods and straight line methods, to derive key aquifer parameters (Cashman and Preene, 2012). Curve fitting methods involve plotting data from each monitoring well on a log-log graph and fitting various theoretical curves to the data until a best fit is found. These methods are time consuming and not straightforward to use. Computer software is usually used to implement these methods.

The straight-line method involves plotting data and a best fit straight line is drawn. This method is a special case of the Theis solution and is based on the work of Cooper and Jacob (1964) and is the most commonly used (Cashman and Preene, 2012). The approach involves plotting drawdown against time since pumping commenced, or drawdown against the distance of the piezometer from the pumping well.

The method selected in this thesis for analysing the pumping test data is the Cooper-Jacob straight line method with the distance-drawdown approach. The data gathered from the piezometers during the constant discharge test were plotted on a semi log graph at a chosen time after pumping commenced. A best fit straight line was then drawn through the data plot. From this graph the slope expressed as Δs, which is the change in drawdown per log cycle of distance, was determined. This method has the added benefit of determining R_o, the distance at which the straight-line intercepts the zero drawdown line. This provides the far field boundary of the pumping test at the time the test was stopped. The drawdown per logarithmic cycle can also be calculated from the same graph.

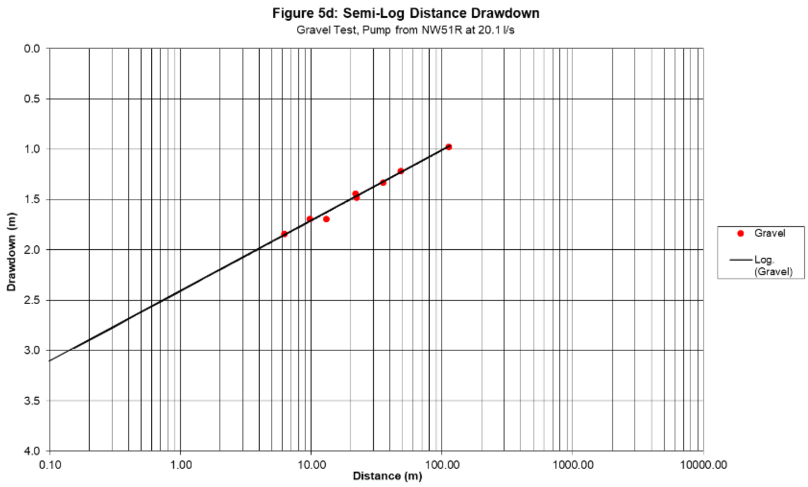


Figure 19 Semi-Log distance drawdown plot for a gravel pumping test pumping at 20.1l/s (North Woolwich, 2010).

The permeability and storage coefficient can then also be obtained from the following equations respectively (Krusemand and De Ridder, 1994):

$$k = \frac{2.3q}{2\pi\Delta sD} \tag{28}$$

$$S = \frac{2.25kDt}{R_0^2} \tag{29}$$

where,

Δs = is the drawdown per log cycle from the distance drawdown plot

t = is the time since pumping commenced (seconds)

R_0 = is the distance of influence

D = is the aquifer thickness

The storage coefficient and permeability can then be calculated, giving:

$S = 0.01$

$K = 1.78 \times 10^{-3}$ m/s

The expected permeability ranges for gravels is in the order of 3×10^{-2} to 3×10^{-4} m/s (Domenico and Schwartz, 1990). The permeability derived from this test falls within this range. As the bottom of the aquifer is not immediately evident, it could be possible that the aquifer thickness was larger than assumed. If an aquifer of double the current thickness is assumed, a permeability value of 8.92×10^{-4} can be derived. However, according to the proposed permeability values by (Domenico and Schwartz, 1990) this value would still be acceptable. The permeability values derived and aquifer thickness assumed is further discussed in chapter 4.

The expected drawdown at the well bore can be estimated by extending the best fit trend line from the distance drawdown plot data to the well bore location. Once the drawdown at the well bore is known, the wetted screen length, h_w , can be calculated subtracting the bottom of the screen depth with the drawdown at the well bore. The drawdown in the aquifer is used rather than the drawdown in the well, as it is assumed that over the seepage face groundwater enters the wells and contributes to the wetted screen length. Although this method of estimating the area over which the flow takes place, is more accurate than past research, a shortcoming is that the well losses, which may occur between the closest monitoring well and the well screen, are ignored. In the event of confined aquifers, it is feasible that the drawdown did not extend to below the screen area and thus no reduction in aquifer thickness occurred. In this case the aquifer thickness remains the same as in the start of the test and is equal to the wetted screen length as shown in Figure 20.

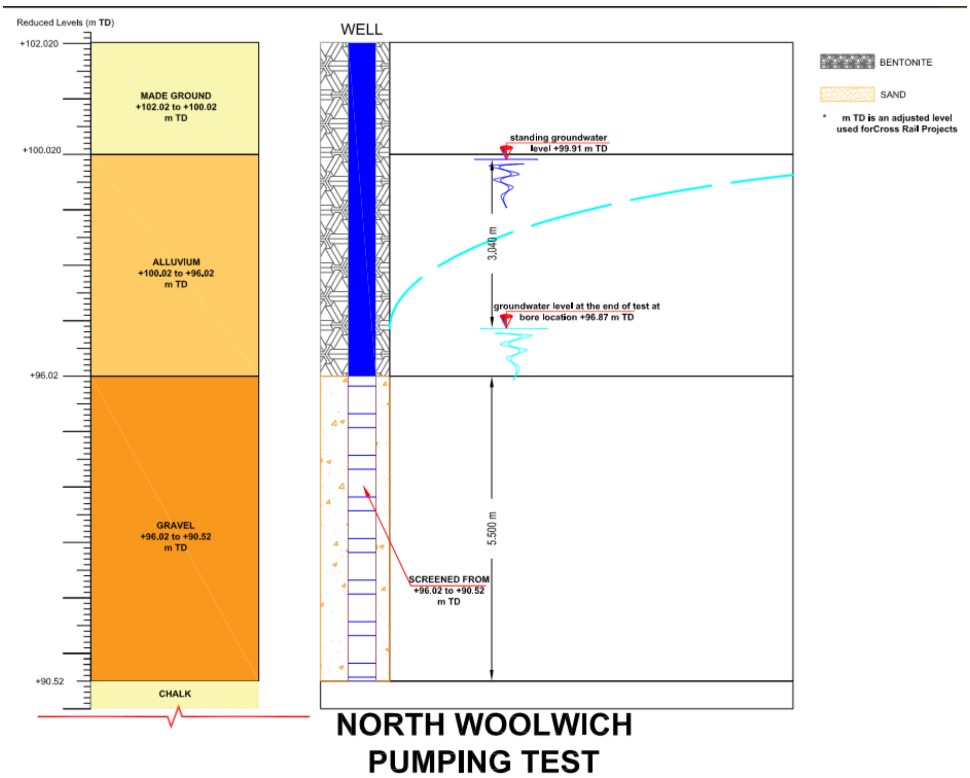


Figure 20: North Woolwich pumping well cross section

The wetted area available for groundwater flow is then:

$$Wetted\ area = h_w\, 2\pi r \tag{30}$$

The implied hydraulic gradient can be calculated using Darcy's law:

$$i = \frac{Q}{Wetted\ area \times k} \tag{31}$$

3.2.2 Belfast Sewers, Ireland (WJGL, 2006)

The main contract works located in Belfast, Ireland consisted of the construction of a terminal pumping station (shaft 16) 40 m in diameter. The confined aquifer consisted of a sandstone from +82.44 mD to +27.44 mD giving an aquifer thickness of 55m. No details on the strata below +27.44mD is available and it is feasible that the sandstone aquifer could continue deeper. The flow regime for the sandstone is expected to be fissured flow, however no mention of this is reflected in the pumping test report. The distance drawdown data also indicates that all the piezometers responded to the pumping and it is feasible that the flow could be mainly matrix flow. The well was pumped at 4.5 l/s during the constant discharge test. The permeability ranges for sandstone is expected to be in the range of 3 x 10^{-10} m/s and maximum values in the order of 6 x 10^{-6} m/s. (Domenico and Schwartz, 1990). The permeability derived from the constant discharge pumping test was 3 x 10^{-6} m/s, which falls within the expected permeability values for sandstone. As the bottom of the aquifer is not immediate evident, it could be feasible that the aquifer thickness was larger than assumed. If an aquifer of double the current thickness is assumed, a permeability value of 1.5 x 10^{-6} can be derived. This assumption would bring the derived permeability values in the ranges proposed by (Domenico and Schwartz, 1990) this value would still be acceptable. This assumption is further discussed in chapter 4.

3.2.3 Uskmouth, New Port pumping test (WJGL, 2007).

In April 2007, WJ Groundwater Limited were appointed to carry out a pumping test at Uskmouth Power Station, Newport, United Kingdom. The main contract works involved construction of a new power station on the site of an old power station, which had been demolished. The site works included the drilling and installation of a single abstraction well. An array of 10 No. standpipe piezometers were monitored to measure the response to pumping. The confined aquifer consisted of sand from -2.9 mD to -14.1 mD giving an aquifer thickness of 11.2m. The bottom of the aquifer is represented by a mudstone layer. The flow regime for the sand is expected to be matrix flow. The well was pumped at 5.5 l/s during the constant discharge test. The permeability ranges for sand is expected to be in the range of 2 x 10^{-7} m/s and maximum values in the order of 5 x 10^{-4} m/s. (Domenico and Schwartz, 1990). The permeability derived from the constant discharge pumping test was 1.2 x 10^{-4} m/s which is in the range suggested by (Domenico and Schwartz, 1990) for fine sand.

3.2.4 Sizewell, Suffolk pumping test (WJGL, 2010).

In March 2010 WJ Groundwater Limited were appointed to carry out a pumping test at the site of the proposed new power station at Sizewell, United Kingdom. The pumping test objectives were to contribute to the general understanding of the hydro-geological conditions within the sand aquifer. The site works included the drilling and installation of a single abstraction well. An array of 14 No. standpipe piezometers were monitored to measure the aquifer response to pumping. The confined aquifer consisted of sand from -9.0 mD to -40.0 mD giving an aquifer thickness of 31m. The bottom of the aquifer is represented by a mudstone layer. The flow regime for the sand is expected to be matrix flow. The well was pumped at 30 l/s during the constant discharge test. The permeability ranges for sand are expected to be in the range of 2 x 10^{-7} m/s and maximum values in the order of 5 x 10^{-4} m/s. (Domenico and Schwartz, 1990). The permeability derived from the constant discharge pumping test was 1.2 x 10^{-4} m/s, which is in the range suggested by (Domenico and Schwartz, 1990).

3.2.5 Vinoli Tower, Abu Dhabi (WJGL, 2007).

In July 2007 Al Naboodah – WJ Groundwater Joint Venture were appointed to carry out a pumping test at Vinoli Tower, Al Raha Beach, Abu Dhabi. The purpose of the pumping test was to investigate the hydraulic characteristics for the design of a dewatering system. The unconfined aquifer consisted of silty sand from +0.19 mD to -9.34 mD giving an aquifer thickness of 9.53m. The bottom of the aquifer is represented by a mudstone layer. The flow regime for the silty sand with is expected to be matrix flow. One fully penetrating deep well and five piezometers screened in the same stratum were used to carry out the test. The sea was located approximately 200m from the pumping well, however as can be noted from the distance drawdown plot, the distance of influence only extends approximately 55m away from the well. Thus, it can be assumed the sea did not affect the distance drawdown during the pumping tests. Furthermore, no tidal influence was observed so no correction to the data was made. The well was pumped at 2.1 l/s during the constant discharge test. The expected permeability ranges for this aquifer is expected to have a permeability range in the order of 1.7×10^{-6} m/s and maximum values in the order of 3.2×10^{-4} m/s. (Rizzo Associates, 2014). The permeability derived from the constant discharge pumping test was 2.78×10^{-5} m/s which falls within this expected permeability range of the sand. (Rizzo Associates, 2014).

3.2.6 Hellevad, Denmark (COWI, 2013).

ENDK commissioned COWI to carry out a test pumping on one of the sites along a pipeline where there was a need for dewatering during construction. The unconfined aquifer consisted of course sand from +33.44 to +19.44 mD giving an aquifer thickness of 14m. The bottom of the aquifer was not proven during the pumping test and no further detailed site investigation was available for this project. The flow regime for the coarse sand is expected to be matrix flow. One deep well and six piezometers screened in the same stratum were used to carry out the test. The well was pumped at 45.69 l/s during the constant discharge test. The expected permeability ranges for the coarse sand is in the order of 9×10^{-7} m/s and maximum values in the order of 6×10^{-3} m/s. (Domenico and Schwartz, 1990). The permeability derived from the constant discharge pumping test was 2.15×10^{-3} m/s which falls within this expected permeability range. (Domenico and Schwartz, 1990). If an aquifer of double the current thickness is assumed, a permeability value of 1×10^{-3} can be derived. This value is still within the permeability ranges suggested by (Domenico and Schwartz, 1990). The assumption of the aquifer thickness for this project is further discussed in chapter 4.

3.2.7 Woolston, Southampton, United Kingdom (WJGL, 2013).

In April 2013 WJ Groundwater Limited were appointed to carry out a pumping test at the site of the existing Woolston wastewater treatment works, near Southampton, Hampshire. The pumping test objectives were to investigate hydrogeological conditions within the silty sand aquifer which can be used for a dewatering system design. The site works included the drilling and installation of a single fully penetrating abstraction well screened within silty sands and an array of six piezometers screened in the same strata. The well was pumped at 0.2 l/s during the constant discharge test. The drawdown generated during the test has been calculated using data at the times of the tidal peak before pumping. The confined aquifer consisted of silty sand from -4.08 to -9.98 mD giving an aquifer thickness of 5.9m. The bottom of the aquifer is represented by a clay layer. The flow regime for the silty sand is expected to be matrix flow. The expected permeability ranges for the silty sand is in the order of 2×10^{-7} m/s and maximum values in the order of 2×10^{-4}

m/s. (Domenico and Schwartz, 1990). The permeability derived from the constant discharge pumping test was 7.76 x 10^{-6} m/s which falls within this expected permeability range. (Domenico and Schwartz, 1990).

Summaries of the pumping test geometry data and the pumping test well details are presented in Table 8 and Table 9 and the derived permeability value for each test is presented in Table 10.

Pumping test reference	Aquifer description	Aquifer boundaries	Top of aquifer (mD)	Bottom of aquifer (mD)	Aquifer thickness (m)
Belfast, Ireland	Sandstone	Confined	82.44	27.44	55
Uskmouth, United Kingdom	Sand	Confined	-2.9	-14.1	11.2
Sizewell, United Kingdom	Sand	Confined	-9	-40	31
***North Woolwich, United Kingdom**	River terrace gravels	Confined	96.02	90.52	5.5
Vinoli Tower, United Arab Emirates	Sand	Unconfined	0.19	-9.34	9.53
Hellevad, Jutland, Denmark	Sand	Unconfined	33.44	19.44	14
Woolston, WTW, United Kingdom	Clayey Sand	Confined	-4.08	-9.98	5.9

Table 8: Aquifer details

Project Name	Ground level (mD)	Depth (m)	Response Zone			Response stratum	Borehole diameter (mm)	Well abstraction rate (l/s)
			Top (mD)	Bottom (mD)	Screen length (m)			
Belfast, Ireland	101.43	74	48.44	27.44	21	Sandstone	300	4.50
Uskmouth, United Kingdom	8.4	23	-2.9	-14.1	11.2	Sand	375	5.50
Sizewell, United Kingdom	2	40	-9	-40	31	Sand	400	30.00
***North Woolwich, United Kingdom**	102	11.5	96.02	90.52	5.5	Gravels	300	20.10
Vinoli Tower, United Arab Emirates	2.06	11.4	0.19	-9.34	9.53	Silty fine sand - reclaimed	425	2.10
Hellevad, Jutland, Denmark	33.44	14	33.44	19.44	14	Sand	400	45.69
Woolston, WTW, United Kingdom	3.32	15.5	-1.68	-12.18	10.5	Clayey Sand	300	0.20

Table 9: Pumping test pumped well detail

Project Location	Duration of pumping test (seconds)	Aquifer thickness (m)	Rest groundwater level (mD)	Groundwater level internal to the well at end of test (mD)	Drawdown achieved internal to the well (m)	Drawdown achieved at well bore location (m)	Distance of influence from distance drawdown plot (m)	Abstraction flow rate (l/s)	ΔS	Transmissivity (m²/s)	Permeability (m/s)	Storage coeficient	Seepage face height
Belfast, Ireland	360,960.00	55	99.07	36.06	63.01	32.26	118	4.5	10.00	1.65E-04	3.00E-06	0.010	30.75
Uskmouth, United Kingdom	270,000.00	11.2	2.16	-10.25	12.41	5.55	220	5.5	1.50	1.34E-03	1.20E-04	0.017	6.86
Sizewell, United Kingdom	864,000.00	31	0.7	-5.69	6.39	4.60	452	30	1.55	7.08E-03	2.29E-04	0.067	1.79
* North Woolwich, United Kingdom	259,200.00	5.5	99.91	95.59	4.32	3.04	2500	20.1	0.75	9.81E-03	1.78E-03	0.001	1.28
Vinoli Tower, United Arab Emirates	432,000.00	9.53	0.19	-8.52	8.71	6.81	55	2.1	2.80	2.75E-04	2.88E-05	0.088	1.90
Hellevad, Jutland, Denmark	172,920.00	14	33.44	31.56	1.88	1.10	200	45.69	0.557	3.00E-02	2.15E-03	0.292	0.78
Woolston, WTW, United Kingdom	274,500.00	5.9	1.2	-7.92	9.12	4.24	85	0.20	1.6	4.58E-05	7.76E-06	0.004	4.88

Table 10: Key aquifer parameters

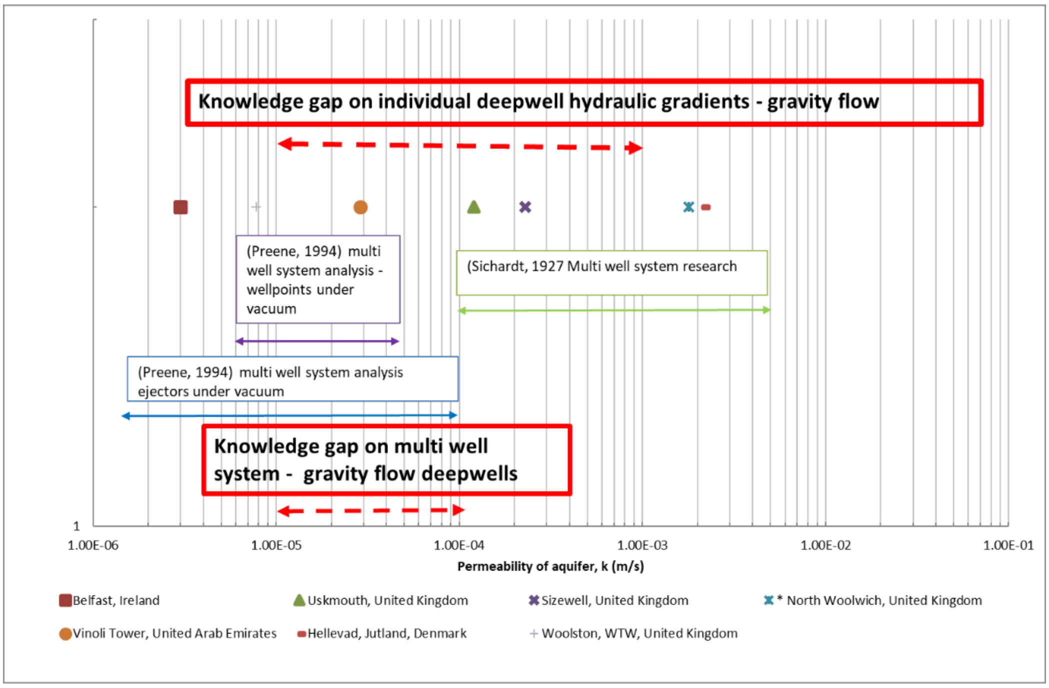


Figure 21: Pumping test permeability settings and the near field boundary knowledge gaps.

Chapter 4: Data analysis

4.1 Single well pumping test data analysis.

To address the knowledge gaps, the new pumping test data have further been analysed. The purpose of the analysis was to derive the implied hydraulic gradient generated during each pumping test, which can then be compared against the current practice and previous research.

The literature review carried out on the previous work of Sichardt (1927) also provided an opportunity to use his data together with modern computer methods to reproduce and verify the results he obtained. This is the first time the Sichardt (1927) data and analysis have been reproduced and verified in this way.

Near field boundary data analysis – implied hydraulic gradient.

The hydraulic gradient for each test can be determined by dividing the abstraction flow rate by the product of the area over which the flow took place and the permeability (Darcy, 1865). The abstraction flow rate for each test is known and the permeability value for each of the pumping tests has been presented in Table 10.
The area over which the flow takes place can be calculated by determining the area of a cylinder:

$$A = 2.\pi.r.lw \qquad (32)$$

with

r = the well radius

lw = the wetted screen over which the flow takes place

The length of wetted screen over which the flow takes place was derived by extending the distance drawdown plot to the well bore location. The total of the wetted screen area of the pumping wells and the derived implied hydraulic gradients are shown in Table 11 .

Project Location	Abstraction flow rate (l/s)	Permeability (m/s)	Wetted screen length at end of constant discharge test (m)	Bore diameter (m)	Flow area (m2)	Hydraulic Gradient (i)
Belfast, Ireland	4.5	3.00E-06	55.0	0.300	51.81	29.00
Uskmouth, United Kingdom	5.5	1.20E-04	10.71	0.375	12.62	3.64
Sizewell, United Kingdom	30	2.29E-04	31.00	0.400	38.94	3.37
*** North Woolwich, United Kingdom**	20.1	1.78E-03	5.50	0.300	5.18	2.18
Vinoli Tower, United Arab Emirates	2.1	2.88E-05	2.724	0.425	3.63	20.06
Hellevad, Jutland, Denmark	45.7	2.15E-03	14.000	0.400	17.58	1.21
Woolston, WTW, United Kingdom	0.2	7.76E-06	5.900	0.300	5.56	4.64

Table 11: New pumping test data implied hydraulic gradients

4.1.1 Pumping test data and current practice

Current best practice for estimating the maximum individual well yields as presented in Preene, et. al. (2000) is a combination of the research carried out by Preene (1994), Preene and Powrie (1993) and Sichardt (1927). In the following analysis, the pumping test data implied hydraulic entry gradients have been plotted against the previous past research.

In Figure 22 the implied hydraulic gradients from the pumping test data have been plotted against the suggestions by Preene (1994) and Preene and Powrie (1993) over a permeability range 1 x 10^{-6} m/s to 2 x 10^{-3} m/s and in Figure 23 presents the same dataset over a permeability range of 1x10^{-6} to 1 x 10^{-4} m/s.

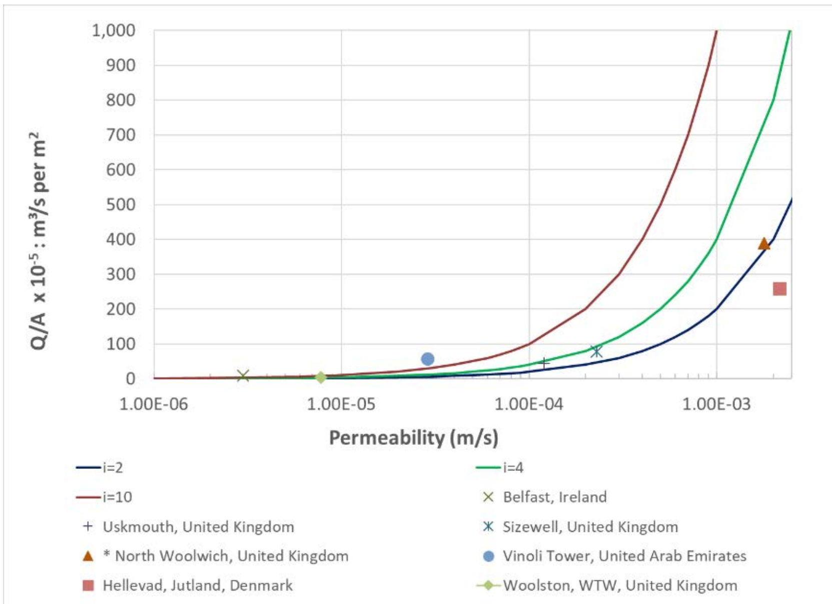


Figure 22: New pumping test data and maximum hydraulic recommendations by Preene (1994)and Preene and Powrie (1993)

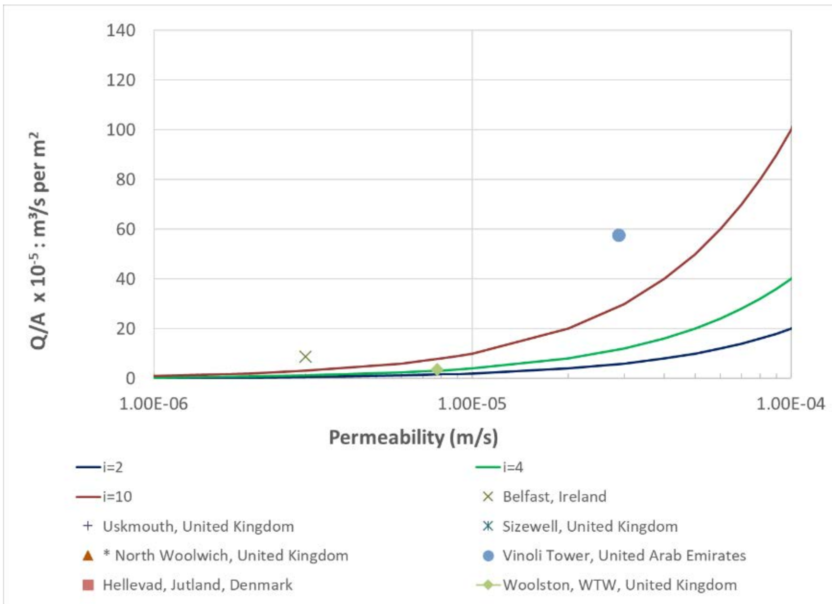


Figure 23: Pumping test data versus Preene (1994) and Preene and Powrie (1993)

When the data is compared to literature, all the data points do not fall within the hydraulic gradient ranges suggested as can be seen on Figure 22 and Figure 23. The data points for Vinoli Tower and Belfast falls above the hydraulic gradient of 10. The data point for Hellevad falls below the hydraulic gradient of 2. For permeability values below 1 x 10^{-4} m/s, the hydraulic gradients of 2 and 4 falls on the lower portion of the data plot while a hydraulic gradient of 10 fits between the data points. Above a permeability of approximately 2 x 10^{-5} m/s the hydraulic gradient of 10 is too high, while the remainder of the data points, except Hellevad, falls between the hydraulic gradients of 4 and 2. The hydraulic gradient for Hellevad is 1.31.

In Figure 24, the pumping test data have been plotted against a fixed hydraulic entry gradient of 6 as suggested by Preene et. al. (2000) over a permeability range between 1 x 10^{-6} up to 1 x 10^{-2} m/s. In Figure 25 the same data set have been plotted over a permeability range of between 1 x 10^{-6} up to 1 x 10^{-4} m/s as suggested by Preene et. al. (2000).

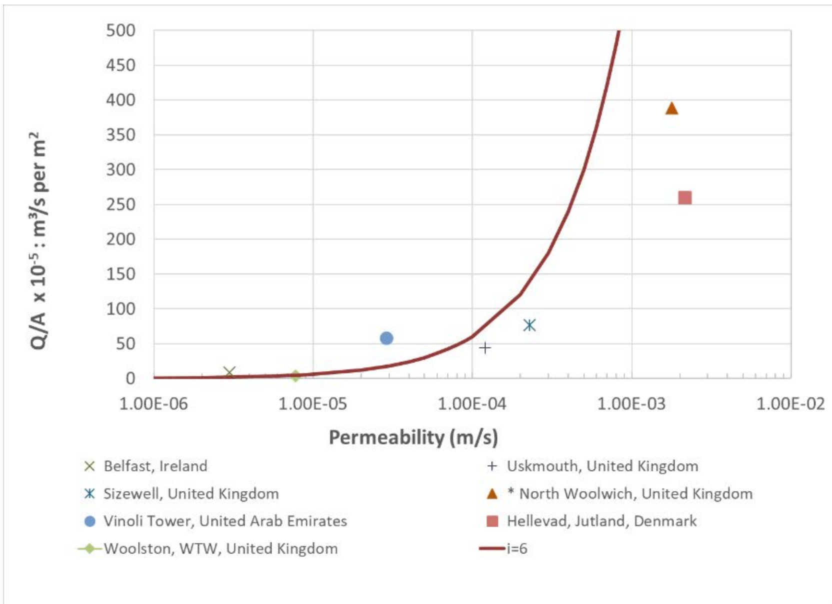


Figure 24: Pumping test data versus i = 6 over a permeability range of 1 x 10^{-6} up to 1 x 10^{-4} m/s

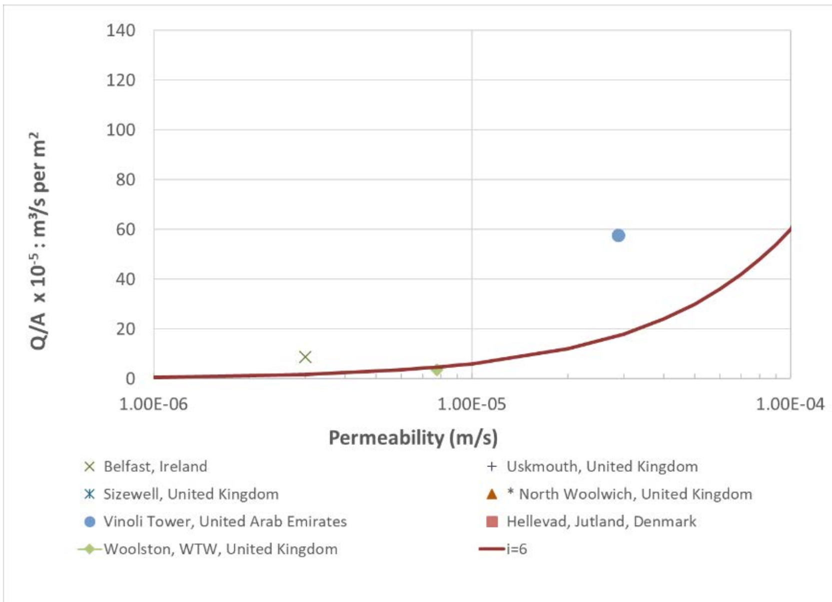


Figure 25: Pumping test data versus recommendations by Preene, et. al. (2000).

When the data is compared to suggestions by Preene, et. al., (2000) on Figure 24 and Figure 25, it can be noted for permeability values below approximately 2 x 10^{-5} m/s a hydraulic gradient of 6 is too low for Belfast and Vinoli and too high for Woolston. For permeability values above 2 x 10^{-5} m/s a hydraulic gradient of 6 is too high. Preene et. al. (2000) also suggests that for permeabilities higher than 2 x 10^{-4} m/s the formula derived by Sichardt (1927) should be used. These criteria have been used to compile Figure 26, which shows the pumping test data and Sichardt's (1927) hydraulic entry gradient criteria over a permeability range of 1 x 10^{-6} m/s up to 1 x 10^{-2} m/s.

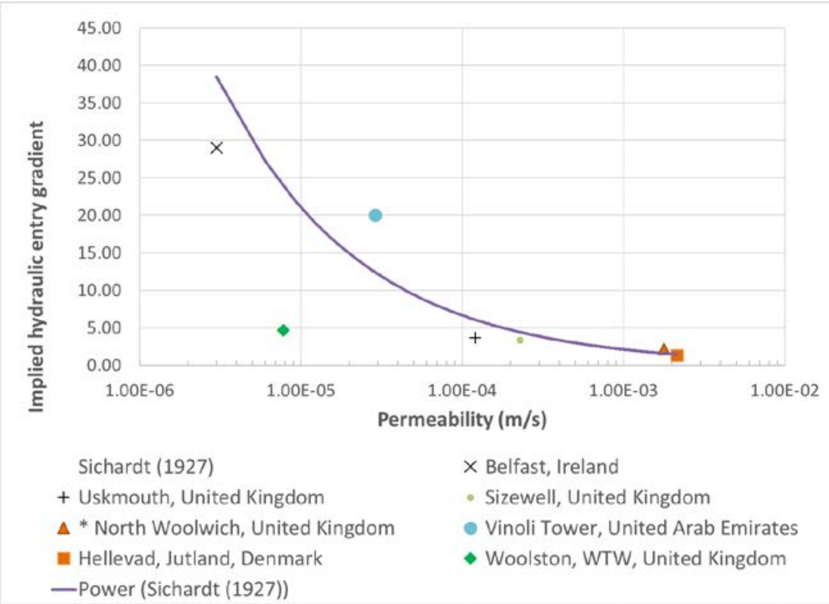


Figure 26: Pumping test data versus Sichardt (1927)

The Sichardt (1927) hydraulic gradient formula graph lies between all the data points over the permeability range of the graph, except for the Hellevad and North Woolwich points, which it intersects at permeability values above 1 x 10^{-3} m/s. The Sichardt (1927) formula appears to fit the data better at permeability values higher than 2 x 10^{-4} than below this value. Considering the general dataset over the range of permeabilities, the Sichardt (1927) formula provides a closer fit than that suggested by Preene (1994), Preene and Powrie, (1993) and Preene, et. al., (2000). Note that the Woolston data point does not follow the general trend of an increased implied hydraulic gradient with a decrease of permeability.

4.1.2 Critical appraisal

A key parameter derived in the analysis of the pumping test data is the permeability values of the aquifers. Once the transmissivity is derived from the pumping test data, the aquifer depth needs to be estimated to calculate the permeability. A common problem in dewatering system design is estimating the aquifer depth (White, 1981). This problem also manifests itself in deriving a permeability values from pumping test data. The rule of thumb proposed to overcome this for dewatering system design is to assume an aquifer depth of 1.5 to 2 times the drawdown required. (Cashman & Preene, 2013). Assuming a deeper aquifer depth during dewatering system design will give a larger required abstraction flow, which will result in a dewatering system with a more robust design with regards to abstraction capacity. Assuming a deeper aquifer during the pumping test analysis will result in a lower permeability value. No definitive criteria are provided in the literature for estimating the aquifer depth during pumping test analysis if the bottom of the aquifer is not immediately available. It might be feasible to use the suggestions by Preene & Cashman (2013) for estimating the aquifer bottom, however as this was proposed in line with dewatering system design, the results should be used with caution.

For the pumping test carried out at Uskmouth, Sizewell, Vinoli Tower and Woolston, the bottom of the aquifer is evident and represented by a stratum deemed to have a low permeability. However, for the pumping tests carried out at North Woolwich, Belfast and Hellevad the bottom of the aquifer is not immediately evident and an estimate had to be assumed.

The permeability value derived for the North Woolwich pumping test was 1.78 x 10^{-3} m/s and falls within the permeability ranges for this stratum (Domenico and Schwartz, 1990). If an aquifer of thickness of two times is assumed, the permeability derived changes to 8.92 x 10^{-4} m/s. This value still falls within the expected permeability ranges for gravels (Domenico and Schwartz, 1990). For this study, it has been assumed that the bottom of the aquifer is represented by the chalk layer present below the gravels, resulting in a permeability value of 1.78 x 10^{-3} m/s.

For the Belfast pumping test, a permeability value of 3 x 10^{-6} m/s was calculated which falls within the permeability ranges which could be expected for sandstone (Domenico and Schwartz, 1990). The aquifer thickness during the pumping test was 55m. If an aquifer of double the current thickness is assumed, a permeability value of 1.5 x 10^{-6} can be derived. This would increase the aquifer thickness to 100m. As the permeability is relatively low, large increases in aquifer depth, 55m in this example, is required to double the permeability value. This value still falls within the expected permeability ranges for sandstone (Domenico and Schwartz, 1990). For this study, it has been assumed that the bottom of the aquifer is represented by the bottom of the well resulting in a permeability value of 3 x 10^{-6} m/s.

The Hellevad pumping test relayed a permeability value of 2.15 x 10^{-3} m/s. Even though this permeability value falls within the ranges which could be expected for sand (Domenico and Schwartz, 1990), there is an uncertainty on the aquifer depth. For permeability calculations in high permeability soils, small increases in the aquifer thickness can significantly affect the permeability derived. As opposed to a 55m aquifer thickness required to halve the permeability on the Belfast pumping test, an aquifer increase of only 14m can have the same effect on the permeability value for this test and a permeability value of 1.07 x 10^{-3} m/s. The derived permeability value for Hellevad is discussed further below.

The pumping test data analysed spans over a permeability range of 7.76×10^{-6} m/s up to 2.15×10^{-3} m/s. This is the first time that hydraulic entry gradients for single wells operating under gravity flow, over a range of permeability values, have been calculated and presented. The upper range for using deepwell systems under gravity flow for dewatering are suggested to be above 1×10^{-5} m/s and below 1×10^{-3} m/s (Preene, et. al., 2000). The permeability values calculated form the pumping test data thus provide insight into the hydraulic gradients for permeability values below and above the conventional permeability ranges over which dewatering systems operating under gravity conditions are suggested. These values could prove useful in the future for special dewatering system designs planned to operate outside the normal operating permeability ranges.

In Figure 22, Figure 23, Figure 24 and Figure 25 which compare the pumping test data against implied hydraulic gradients form Preene (1994), Preene and Powrie (1993) and Preene, et. al. (2000), it can be noted that none of these proposed hydraulic gradients sufficiently provide a good fit to the entire dataset. Attention is drawn to the Vinoli data point, which indicates a higher yield per m^2 compared to Uskmouth, even though the Uskmouth data point has a much higher permeability value. This could be due to the Vinoli aquifer being unconfined as well as the reduction of the aquifer thickness close to the well when pumping. This will result in a smaller area available for the groundwater to enter the well and hence a higher hydraulic gradient. On most of the confined aquifers no reduction in aquifer thickness and wetted screen took place. Likewise, the Hellevad data point indicates that this data point has a lower yield per m^2 of wetted screen even though it has the highest permeability. Considering the uncertainty of the Hellevad aquifer thickness, it is feasible that the thickness is underestimated which resulted in too high a permeability values derived, possibly explaining this anomaly. Using an aquifer thickness of two times the well depth, a permeability value of 1.07×10^{-3} m/s can be derived. In Figure 27 the adjusted Hellevad data point has been plotted along with the other pumping test data points versus the hydraulic gradient suggestion by Preene, et. al. (2000).

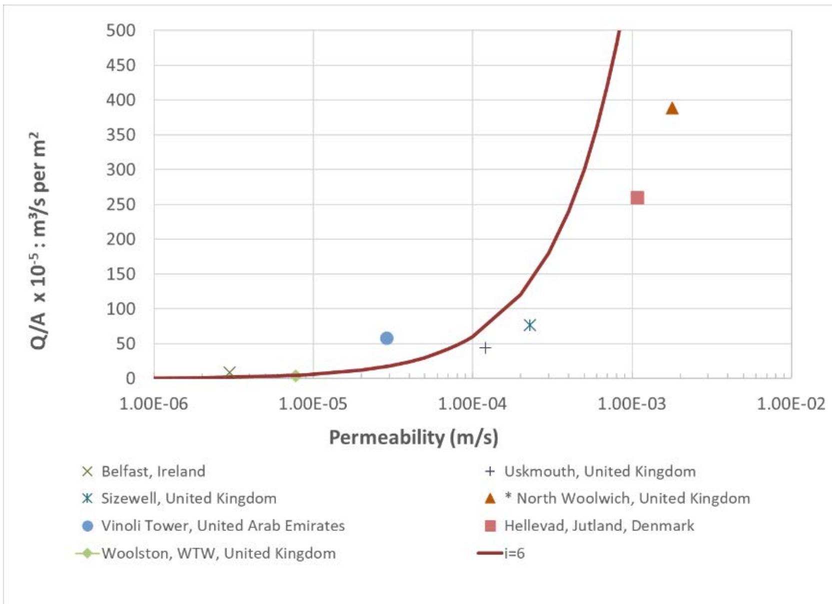


Figure 27: Pumping test data, with adjusted Hellevad, versus Preene, et. al., (2000).

It can be noted that the Hellevad point now follows the general trend where a reduction in permeability reflects a reduction in abstraction flow per m^2 of wetted screen and vice versa, explaining the increase in aquifer thickness suggestion.

Using the dataset in Figure 27, alternative hydraulic gradient of 3 is proposed and presented in Figure 28.

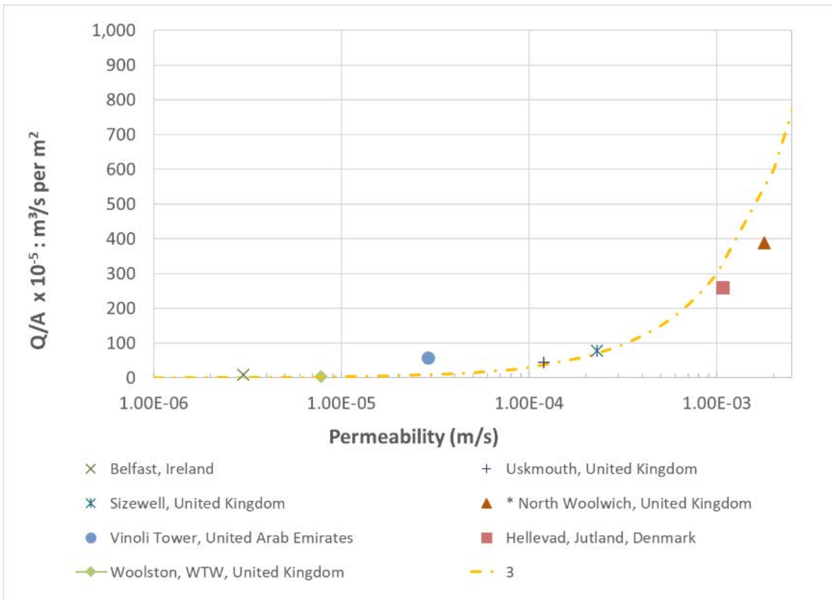


Figure 28: Pumping test data versus hydraulic gradient of 3

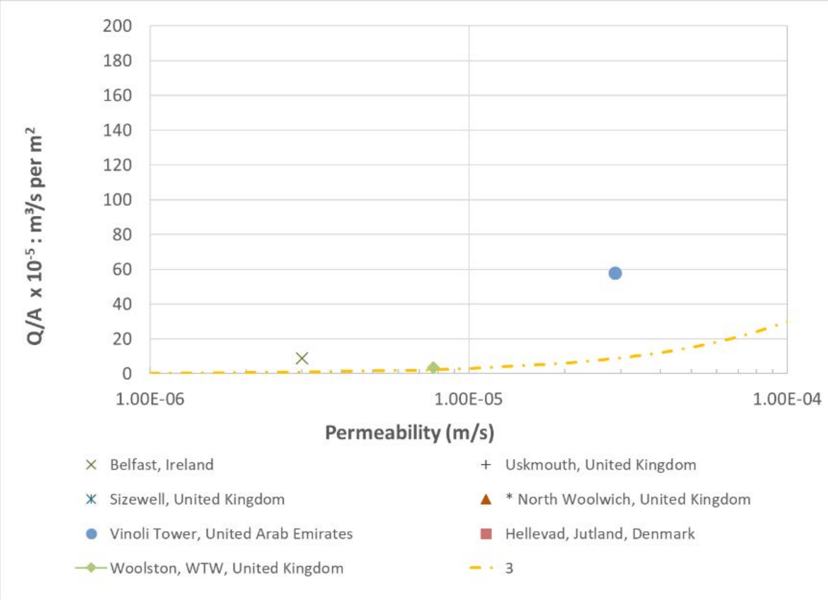


Figure 29: Pumping test data versus hydraulic gradient of 2.5 for a permeability range of 1 x 10^{-4} to 1 x 10^{-4} m/s

It can be noted that at the low permeability ranges the hydraulic gradient of 3 appears to be too low, however provides a good fit to the data points for the remainder of the data points. It should also be considered that most dewatering systems under gravity operate in permeability values higher than 1 x 10^{-5} m/s. (Preene, et. al., 2000) where this value provides a good fit.

The comparison in Figure 26 shows that the Sichardt (1927) formula provides a good fit for the data above 2 x 10^{-4} m/s. The data point for Woolston appears to have a lower hydraulic gradient than the general trend for the other data points. It is not immediately evident why the hydraulic gradient is lower than expected, other than that the abstraction flow was low, 0.2 l/s, and the pumping test took place in a low permeability setting, 7.76 x 10^{-6} m/s. From the above analysis, it is evident that not one hydraulic entry gradient value sufficiently represents a large range of permeability values. The Sichardt (1927) formula does however fit the entire dataset better than a single fixed hydraulic gradient. Even though the proposed hydraulic gradient of 3 appears to be a good fit, at permeability values lower than 1 x 10^{-5} and higher than 2 x 10^{-3} the Sichardt (1927) formula provides a better fit on the overall dataset over the range of permeability values.

4.1.3 Pumping test data and verification of Sichardt (1927)

In this section, the data gathered during the literature review on Sichardt's (1927) research is re-analysed, presented and evaluated. The data have been reproduced based on Sichardt's (1927) original datasets and presented in Figure 30. Added is a graph representing Sichardt (1927) derived formula.

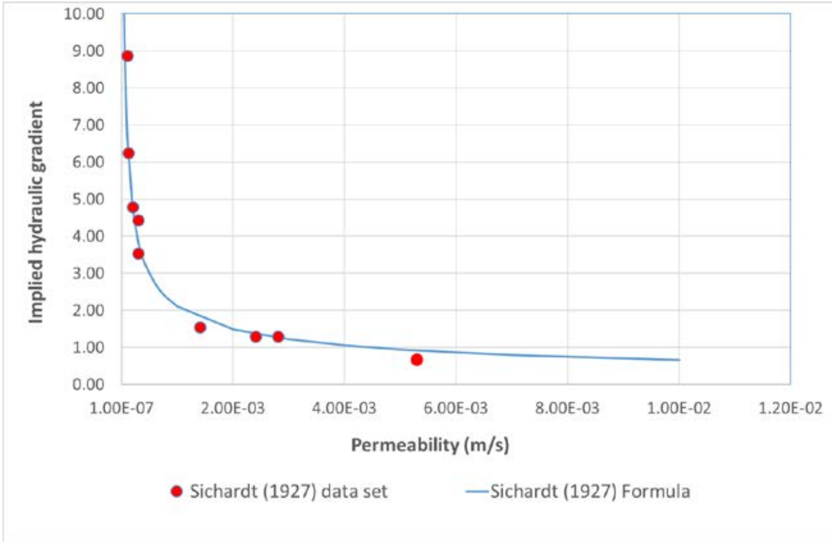


Figure 30: Sichardt (1927) data and Sichardt (1927) derived formula for estimating hydraulic gradients.

The permeability range over which Sichardt (1927) derived his formula, 1.03 x 10^{-4} m/s to 5.3 x 10^{-3} m/s, is generally focussed to the higher end of permeability values over which gravity dewatering takes place. It can be noted that the Sichardt (1927) formula provides a good fit for the data points, except the lowest permeability data points for a permeability value of 1.03 x 10^{-4} m/s. Little detailed data is available on the exact geological setting in which Sichardt data was collected. The details are limited to stating they are “sand” aquifers (Sichardt, 1927). No further classification of the aquifer is presented in his research and little else is known on how he derived his permeability values except that it has been back calculated from operated dewatering systems. The expected permeability for course sand is 9 x 10^{-7} to 6 x 10^{-3} m/s (Domenico and Schwartz, 1990). This is the first time in literature that Sichardt (1927) formula has been validated and reproduced. This provides some insight on the history of this formula, the geological setting his data was collected in and the range of permeabilities it expanded over.

Chapter 5: Conclusion

A review of the literature on the boundary conditions of groups of wells and the shortcomings of the current practices used for estimating individual well yields are presented in chapter 2. In chapter 3, new single well pumping test data have been presented and used to calculate the implied hydraulic gradients at entry into the pumping wells. In chapter 4 the new data analysis results have been compared against the current practices and guidance is given on estimating individual well yields. Also in chapter 4, the Sichardt (1927) research has been validated and reproduced for the first time.

This chapter concludes with reasoned recommendations to modify the guidance on estimating of the near field boundary conditions of individual wells operating under gravity conditions.

5.1 Near field boundary

Previously unpublished data in the form of single well pumping tests have been used to calculate the bulk horizontal permeability in which the pumping tests took place, the area over which the flow entered the well, as well and the hydraulic entry gradient to each individual well. Existing methods previously published in the literature have been used to calculate the permeability values based on single pumping test data. However, this is the first time that single well pumping test data for wells operating under gravity conditions have been analysed using the distance drawdown plot to estimate the implied hydraulic gradient. The data analysed also presents a significant range of permeability values extending between 7.76 x 10^{-6} m/s up to 1.78 x 10^{-3} m/s. Considering that gravity dewatering is suggested over ranges between 1 x 10^{-5} m/s up to 1 x 10^{-3} m/s by Preene, et. al., (2000), the range of gravity fed pumping tests analysed, extends above and below this. This could greatly assist in the design of special dewatering systems where the there is a need to push the boundaries of the conventional range over which gravity dewatering takes place.

Using the pumping test data, the implied hydraulic gradient for each test has been calculated and presented in Table 11. The plots related to Figure 22, Figure 24 and Figure 26 indicate that the implied hydraulic gradient values when plotted against the relevant permeability value, do not all comply with the suggestions by Preene and Powie (1993), Preene (1994) and Preene, et. al (2000). It can be concluded that the current best practice guidelines do not provide sufficient recommendations for deepwells operating under gravity flow conditions. Also, the formula presented by Sichardt (1927), although it was based on groups of wells operating in a relatively high permeability setting, provides a reasonable fit for a larger permeability rage when compared against other recommendations in the literature.

For the lower range of permeabilities, in the order of approximately below 1 x 10^{-5}, the research does not provide sufficient data to estimate a single viable hydraulic gradient with any confidence for wells operating under gravity flow conditions. The Sichardt (1927) formula for estimating hydraulic gradient provides the best fit for the data over a large range of permeability values when compared to suggestions by Preene and Powrie (1993), Preene (1994) and Preene, et. al. (2000).

Figure 31 has been compiled for possible use at first estimation of individual well yields based on the hydraulic gradients based on Sichardt (1927). It shows the relationship between aquifer permeability and well yield per unit length of wetted screen per unit effective well radius. This figure can be used to provide a first estimate of average individual well yields, but should not be relied upon until supported by appropriate practical experience. For systems operating in low permeability settings it is suggested to carry out a pumping test to investigate well yields or rely on practical experience where applicable.

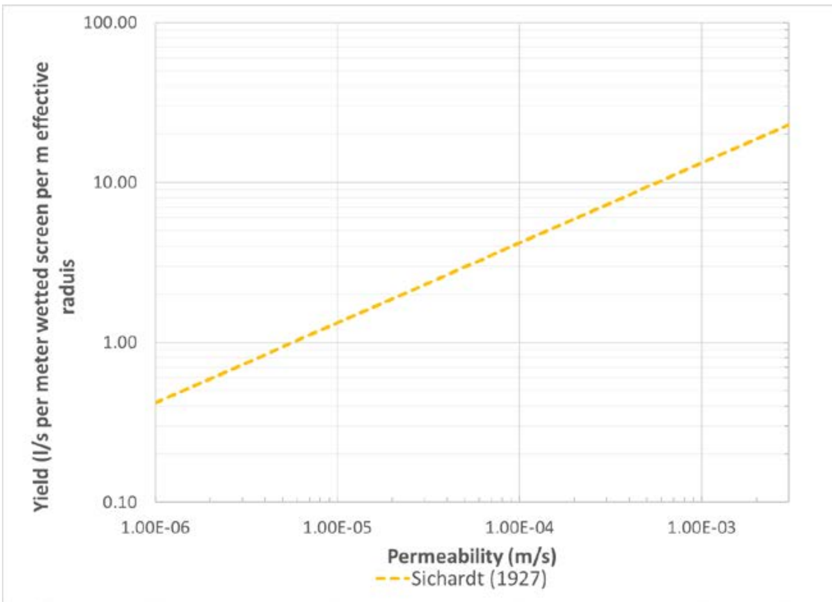
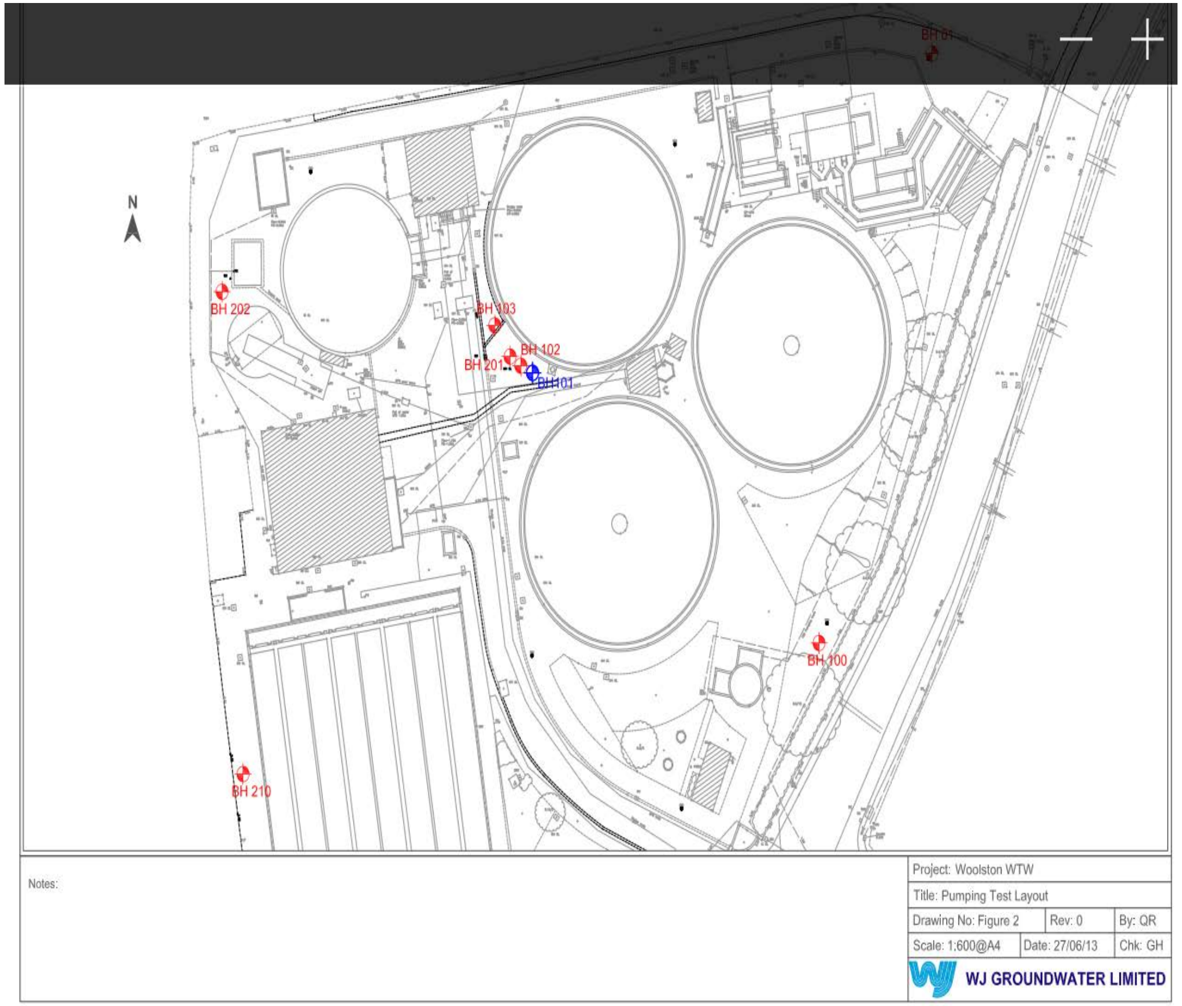


Figure 31: New suggested hydraulic entry gradients into individual wells.

Appendices

Appendix A

Woolston WTW



Appendix A

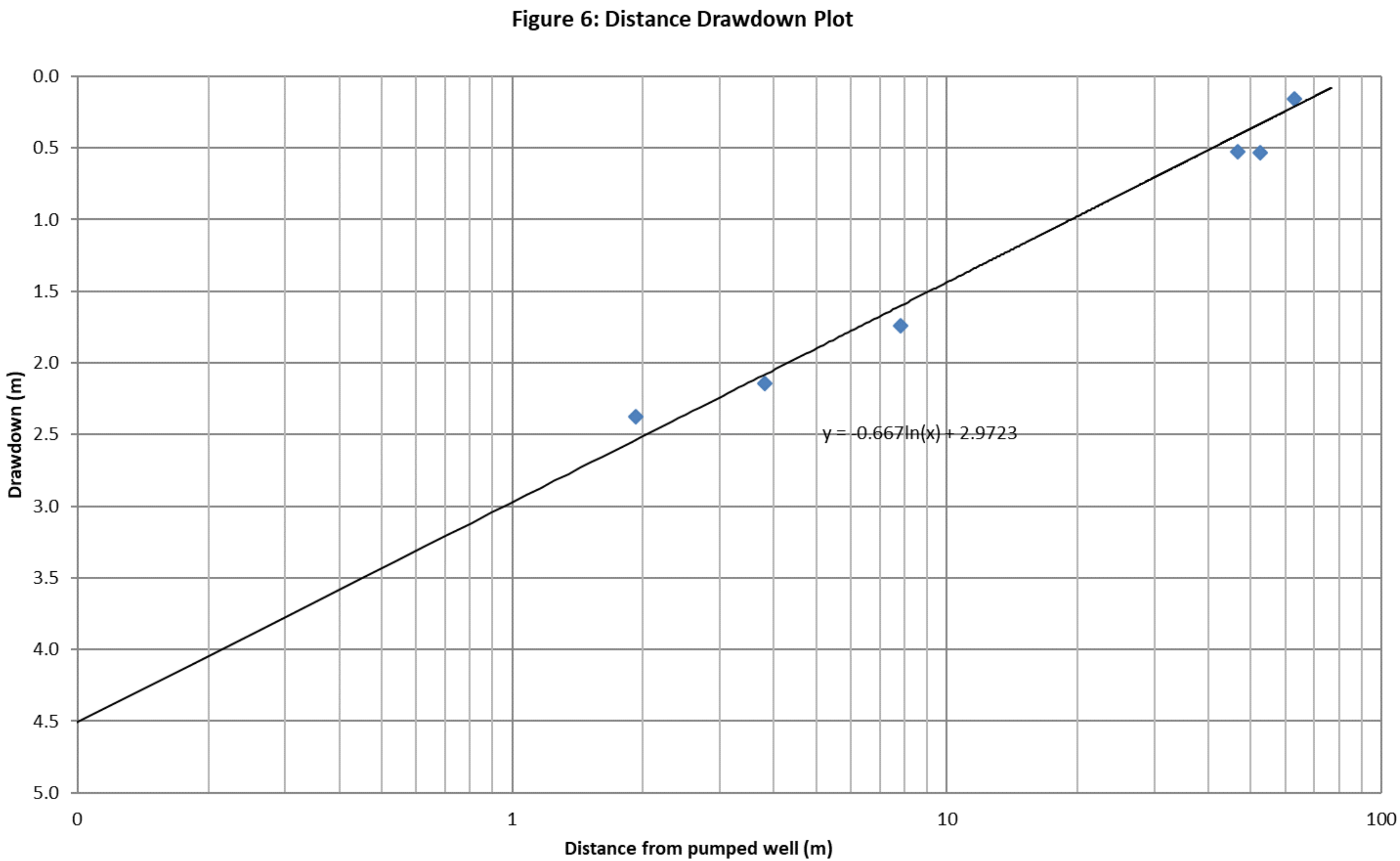
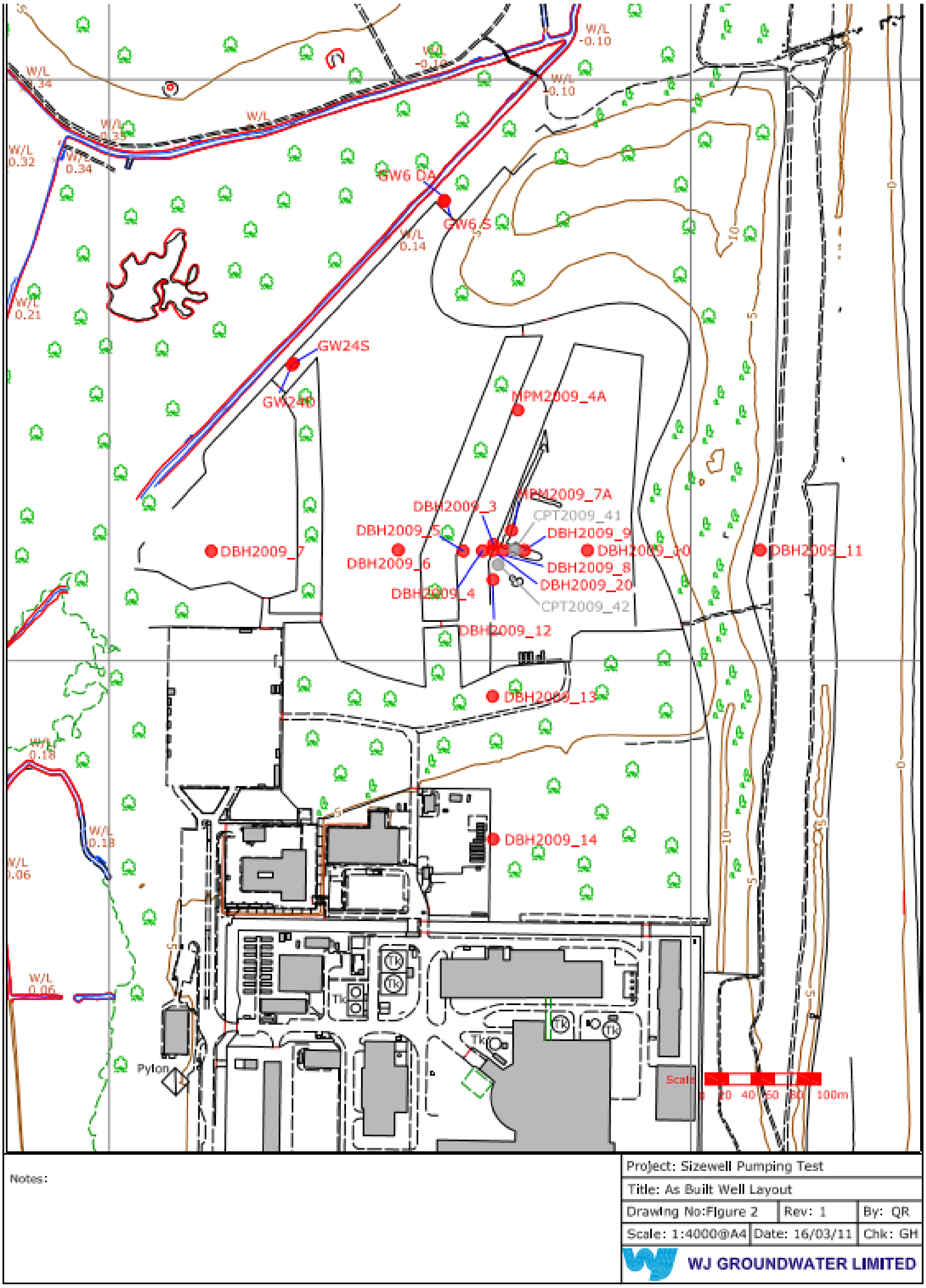


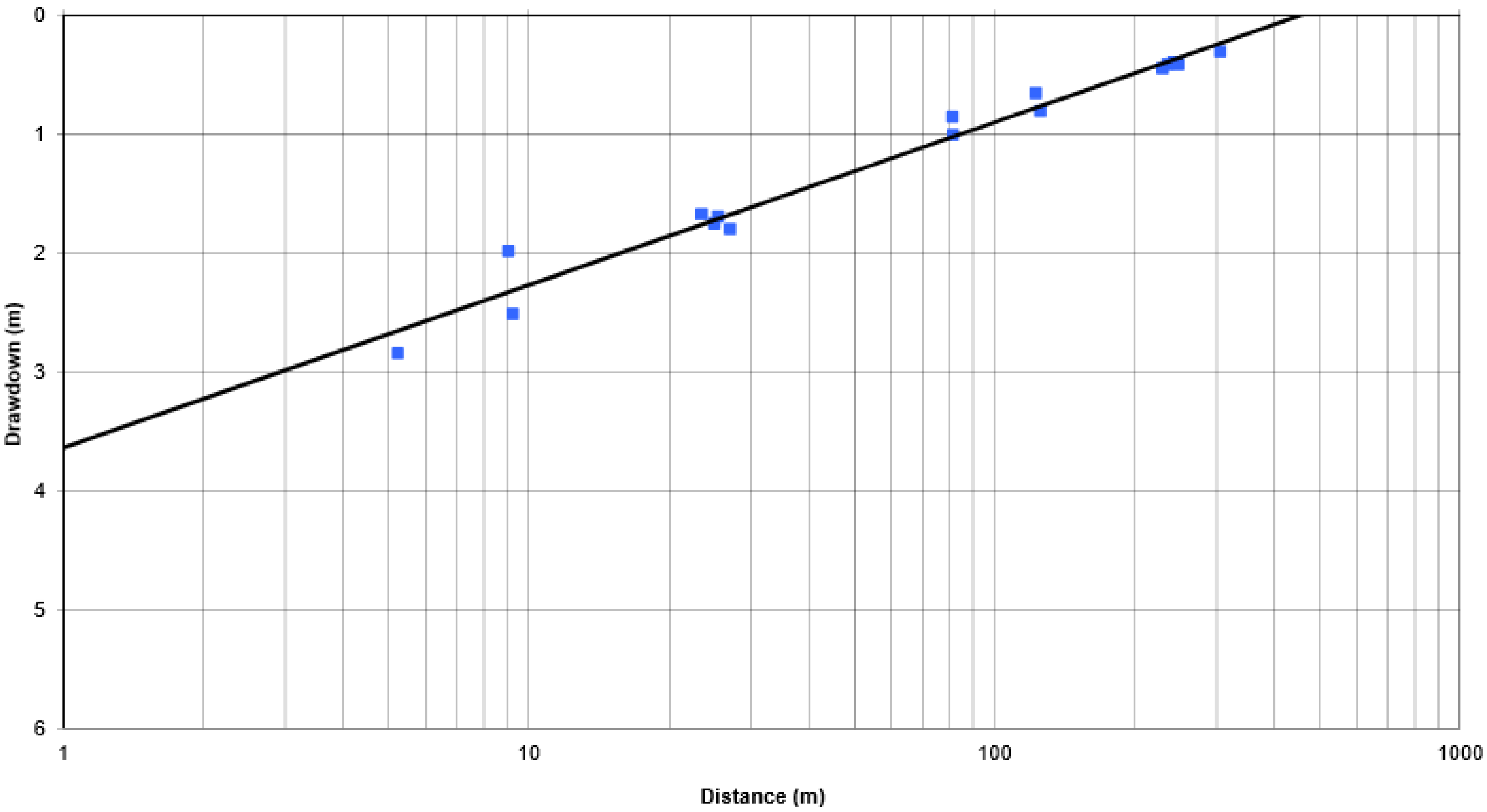
Figure 6: Distance Drawdown Plot
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
Drawdown (m)
y = -0.667ln(x) + 2.9723
0
1
10
100
Distance from pumped well (m)

Sizewell

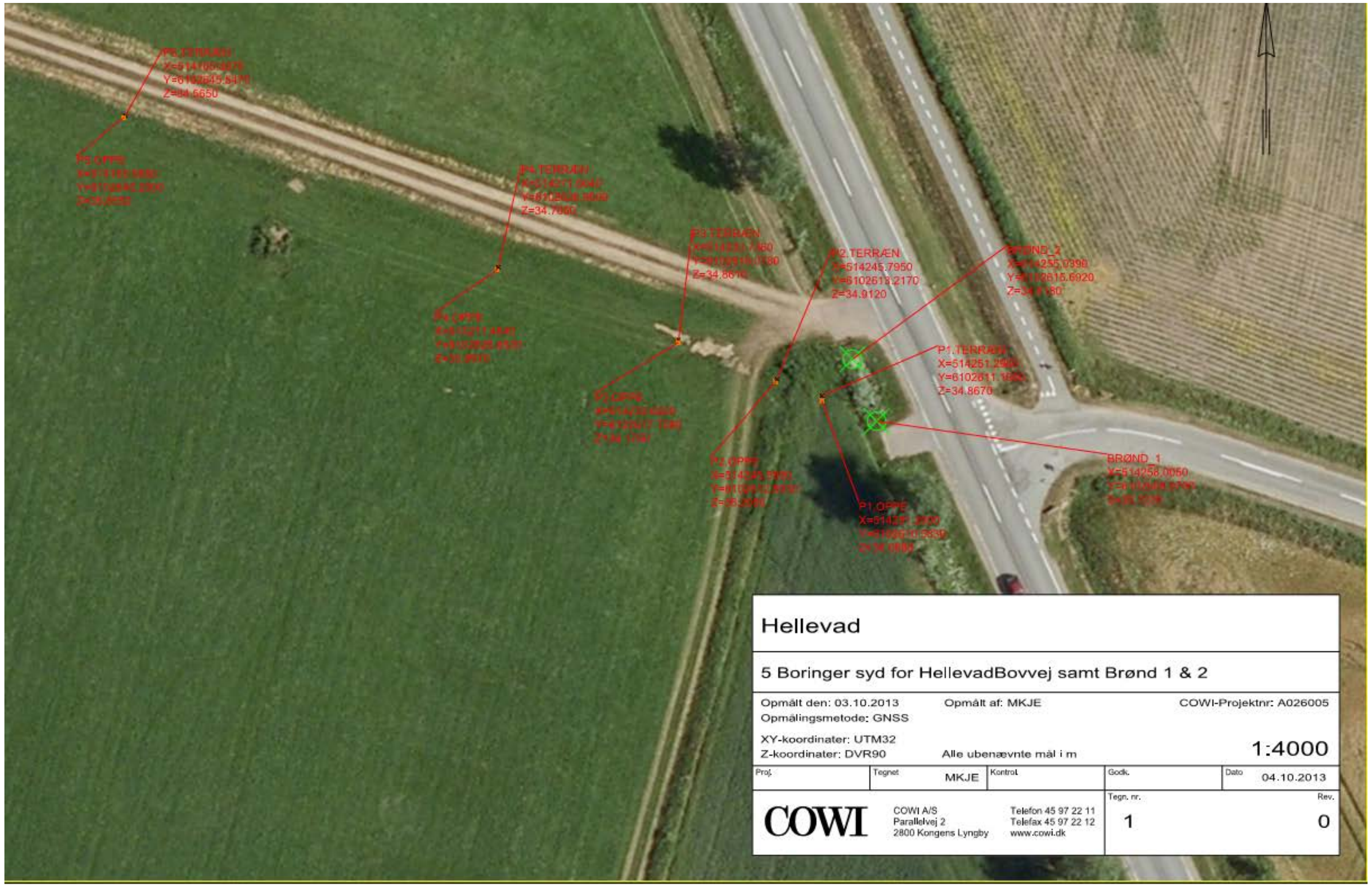

GW6 DA
GW6 S
GW24S
GW24D
MPM2009_4A
MPM2009_7A
CPT2009_41
DBH2009_3
DBH2009_5
DBH2009_9
DBH2009_7
DBH2009_6
DBH2009_10
DBH2009_11
DBH2009_8
DBH2009_20
DBH2009_4
CPT2009_42
DBH2009_12
DBH2009_13
DBH2009_14
Pylon
Scale
20 40 60 80 100m
Notes:
Project: Sizewell Pumping Test
Title: As Built Well Layout
Drawing No:Figure 2
Rev: 1
By: QR
Scale: 1:4000@A4
Date: 16/03/11
Chk: GH
WJ GROUNDWATER LIMITED

WJ GROUNDWATER LIMITED
J1638 Sizewell Power Station
Pumping Test

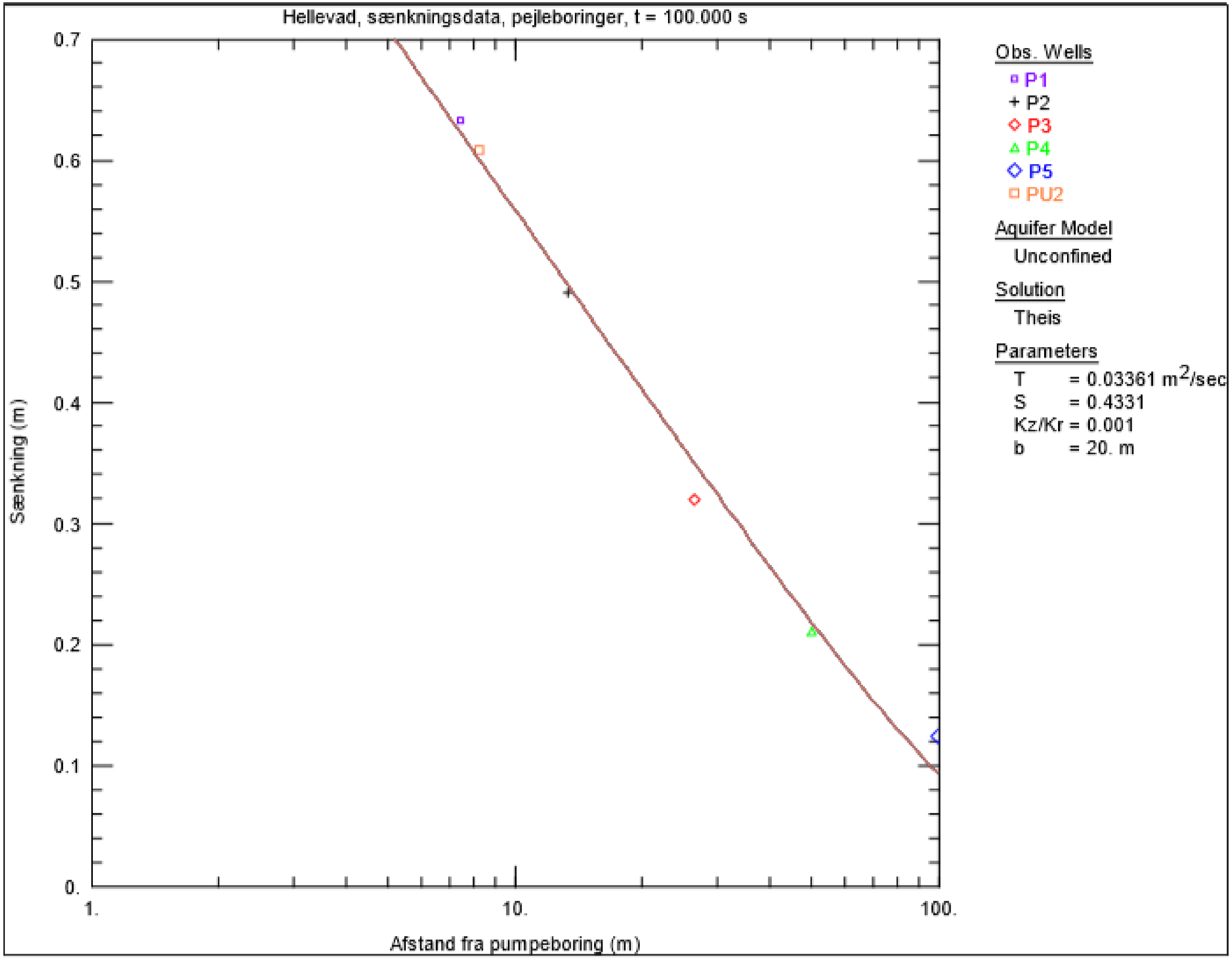
Figure 6: Distance Drawdown Plot



Hellevad

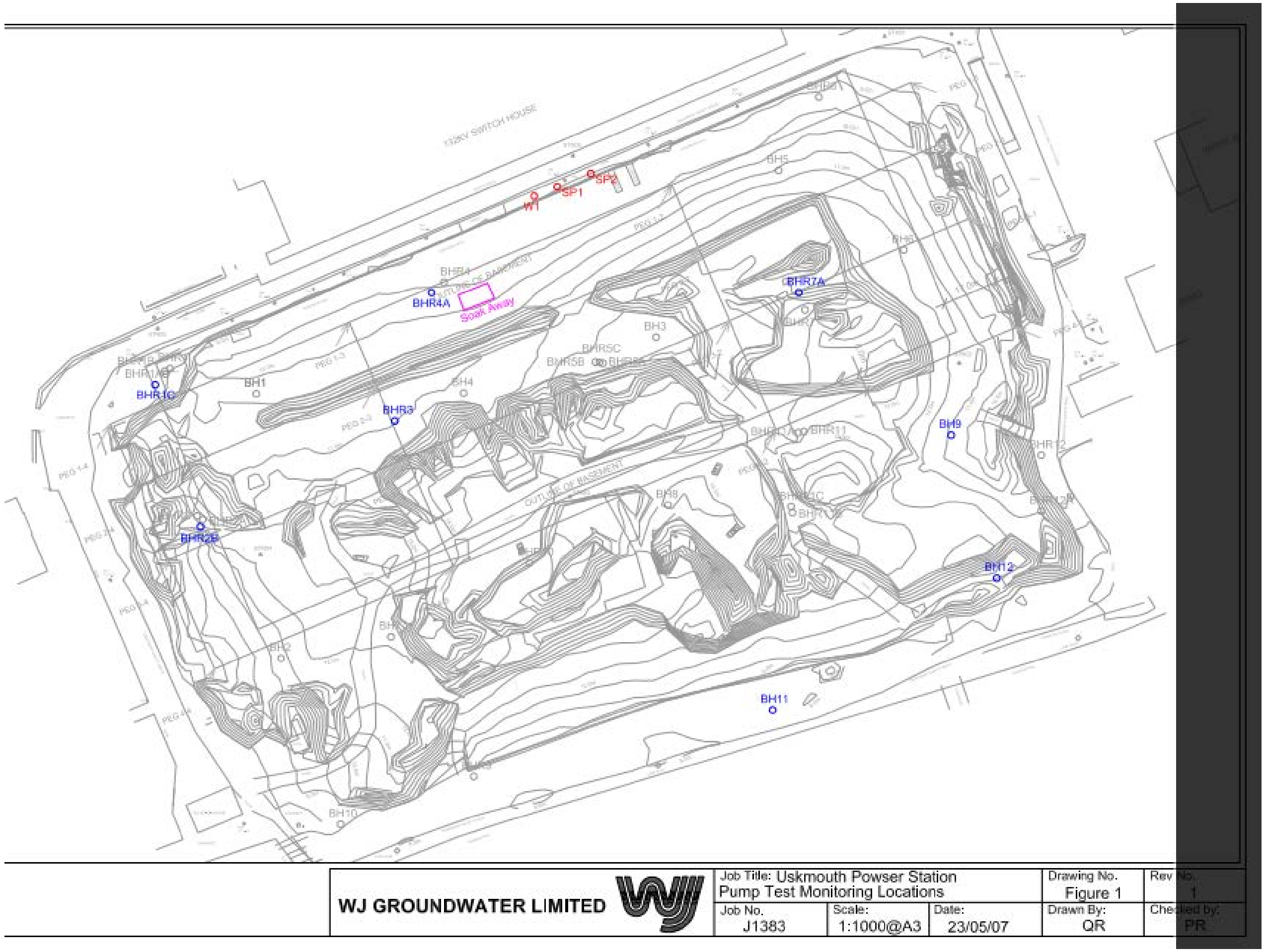

P2.TERRÆN
Y=6102613.2170
Z=34.9120
P1.TERRÆN
Z=34.8670
P1.OPPE
BRØND_1
Hellevad
5 Boringer syd for HellevadBovvej samt Brønd 1 & 2
Opmålt den: 03.10.2013
Opmålt af: MKJE
COWI-Projektnr: A026005
Opmålingsmetode: GNSS
XY-koordinater: UTM32
Z-koordinater: DVR90
Alle ubenævnte mål i m
1:4000
Proj.
Tegnet
MKJE
Kontrol
Godk.
Dato
04.10.2013
COWI
COWI A/S
Parallelvej 2
2800 Kongens Lyngby
Telefon 45 97 22 11
Telefax 45 97 22 12
www.cowi.dk
Tegn. nr.
1
Rev.
0

Appendix A



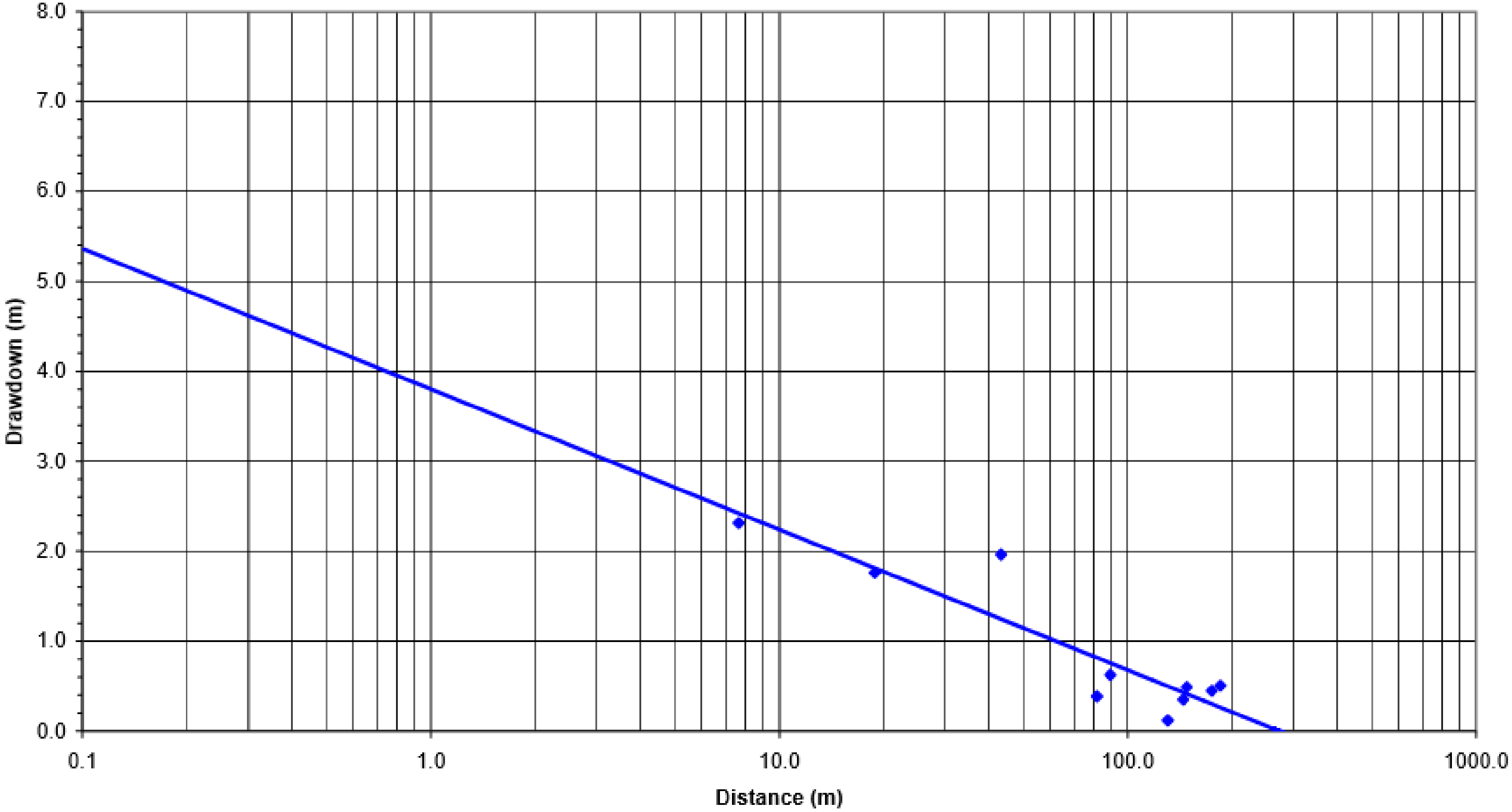
Hellevad, sænkningsdata, pejleboringer, t = 100.000 s
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.
1.
10.
100.
Sænkning (m)
Afstand fra pumpeboring (m)
Obs. Wells
P1
P2
P3
P4
P5
PU2
Aquifer Model
Unconfined
Solution
Theis
Parameters
T = 0.03361 m2/sec
S = 0.4331
Kz/Kr = 0.001
b = 20. m

Uskmouth

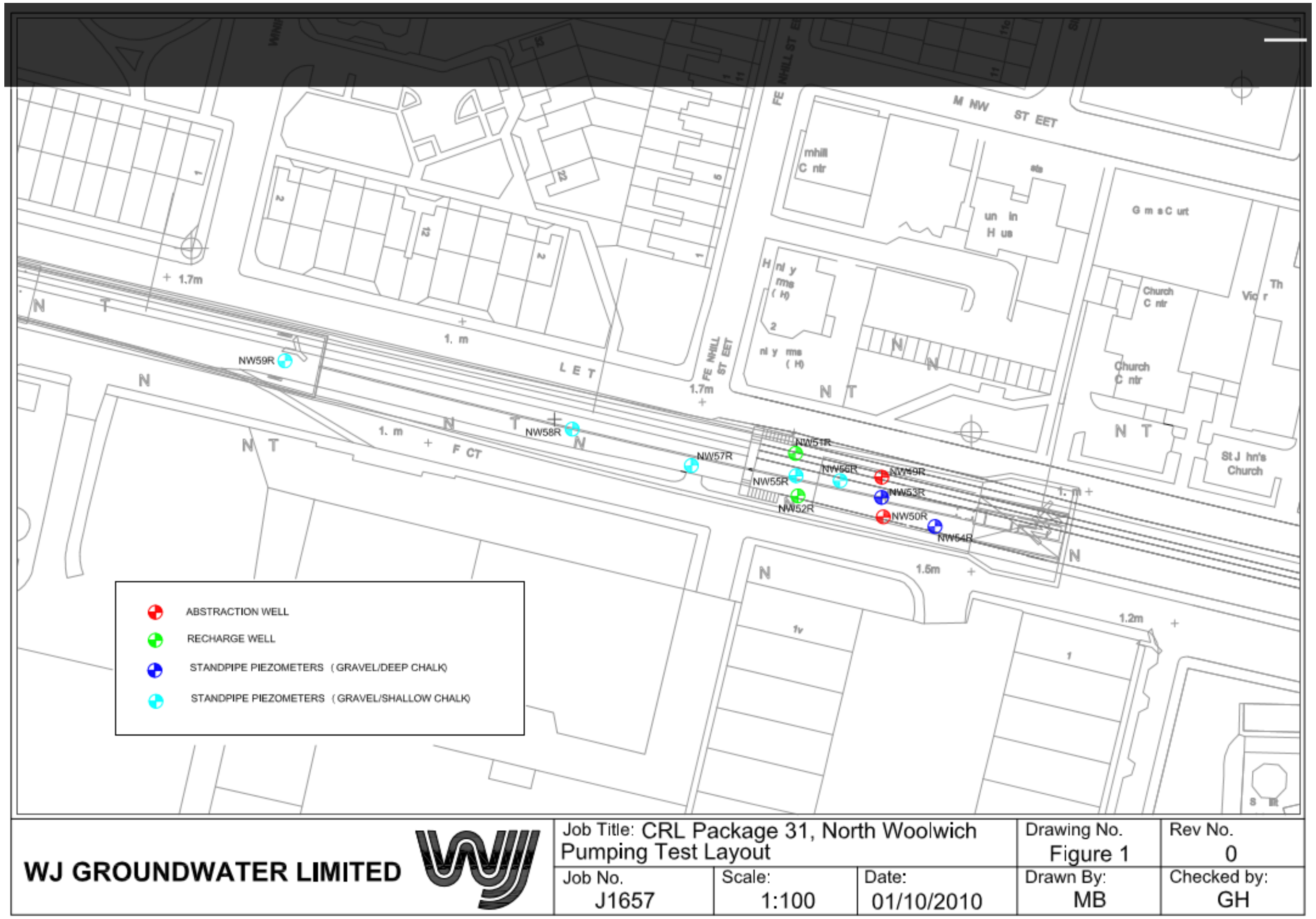

132kV SWITCH HOUSE
W1
SP1
SP2
BHR4A
Soak Away
BHR7A
BHR3
BHR1C
BHR2B
BH9
BH12
BH11
BH1
BH3
BH4
BH5
BH10
WJ GROUNDWATER LIMITED
Job Title: Uskmouth Powser Station
Pump Test Monitoring Locations
Job No.
J1383
Scale:
1:1000@A3
Date:
23/05/07
Drawing No.
Figure 1
Drawn By:
QR
Rev No.
1
Checked by:
PR

WJ GROUNDWATER LIMITED
J1383 Uskmouth Power Station
Pumping Test

Figure 4: Semi-Log Distance Drawdown Plot



North Woolwich



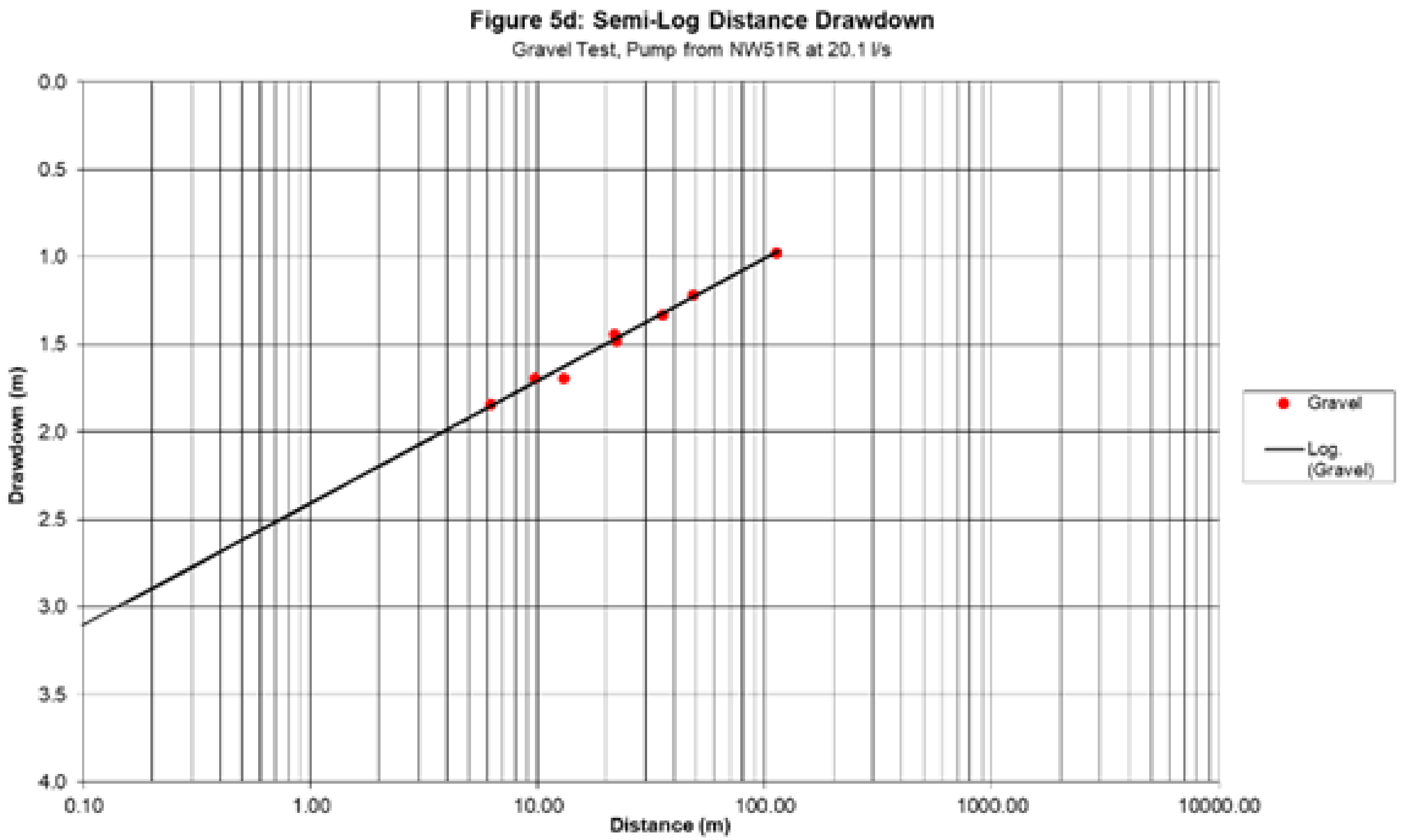
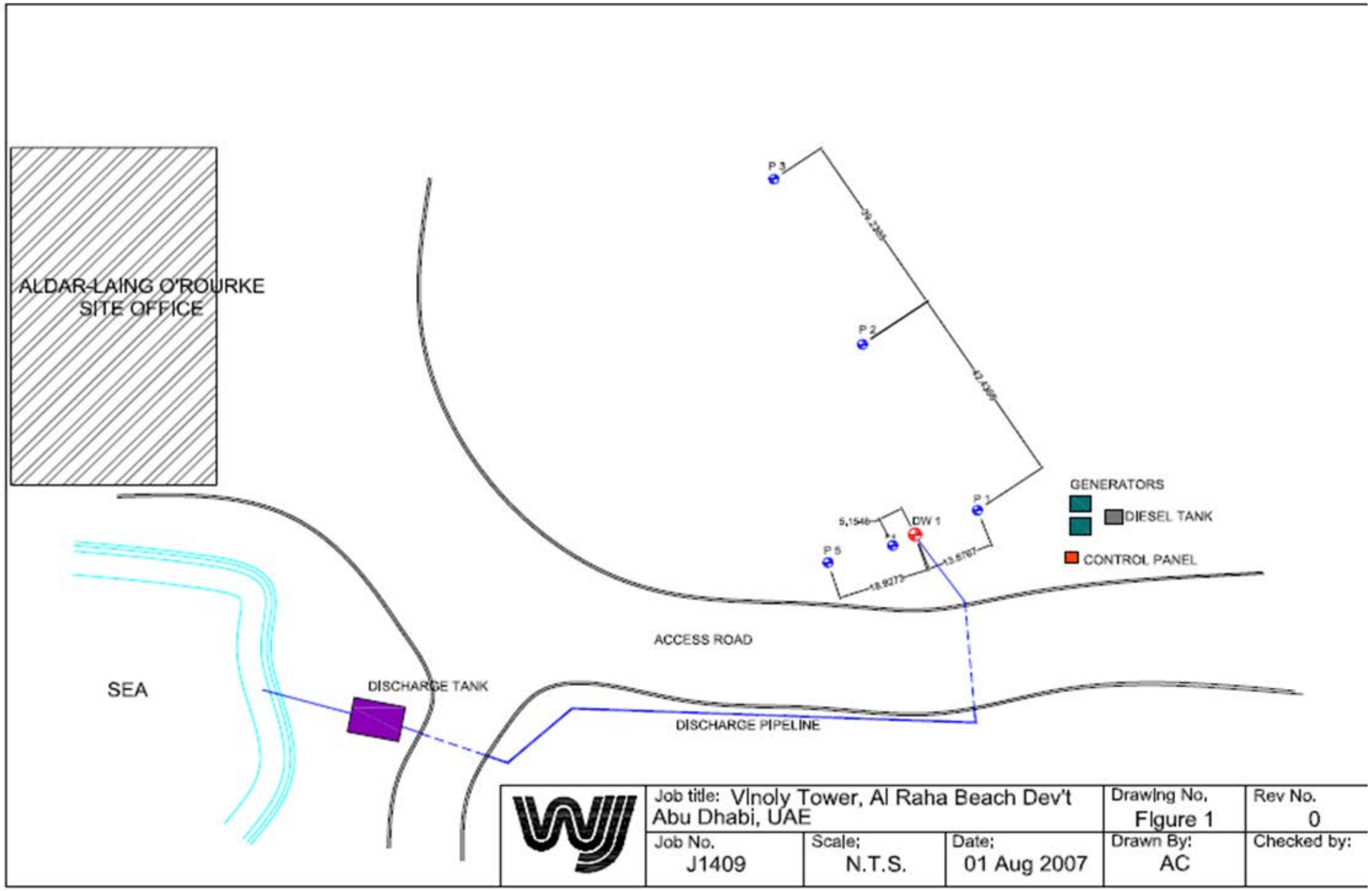


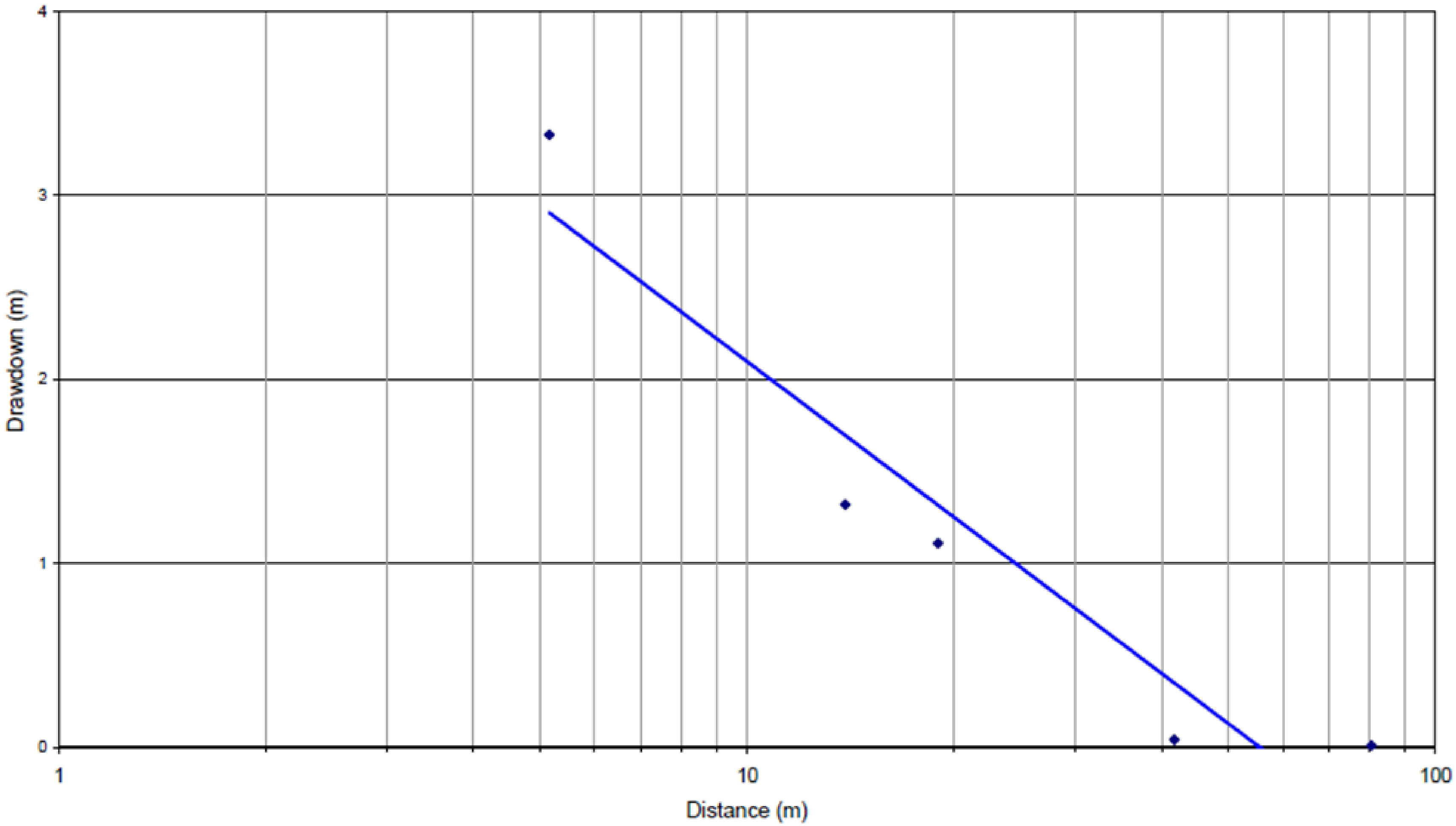
Figure 5d: Semi-Log Distance Drawdown
Gravel Test, Pump from NW51R at 20.1 l/s
Drawdown (m)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
0.10
1.00
10.00
100.00
1000.00
10000.00
Distance (m)
Gravel
Log. (Gravel)

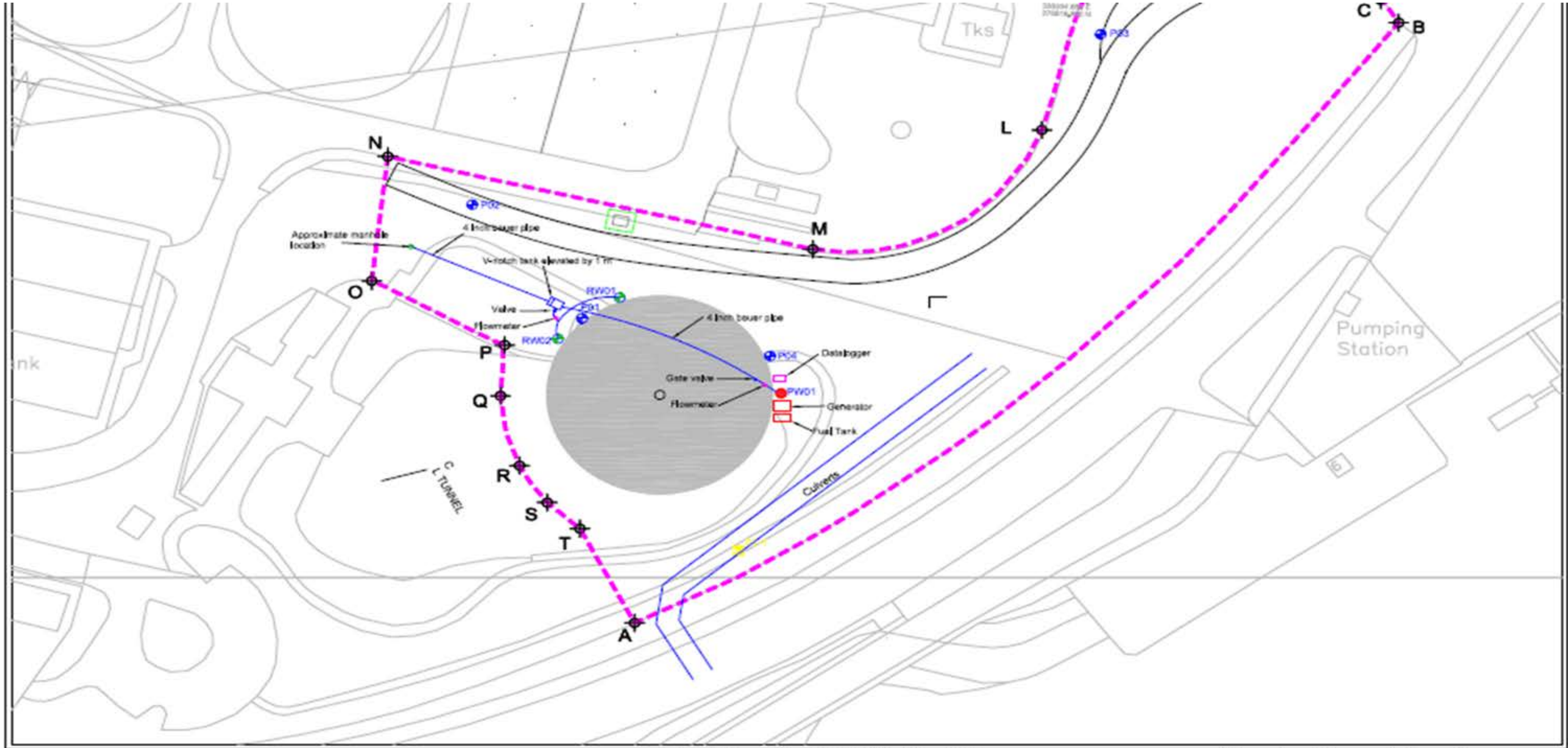
Vinoli Tower



WJ GROUNDWATER LIMITED
Vinoly Tower, Al Raha Beach Development
Pumping Test

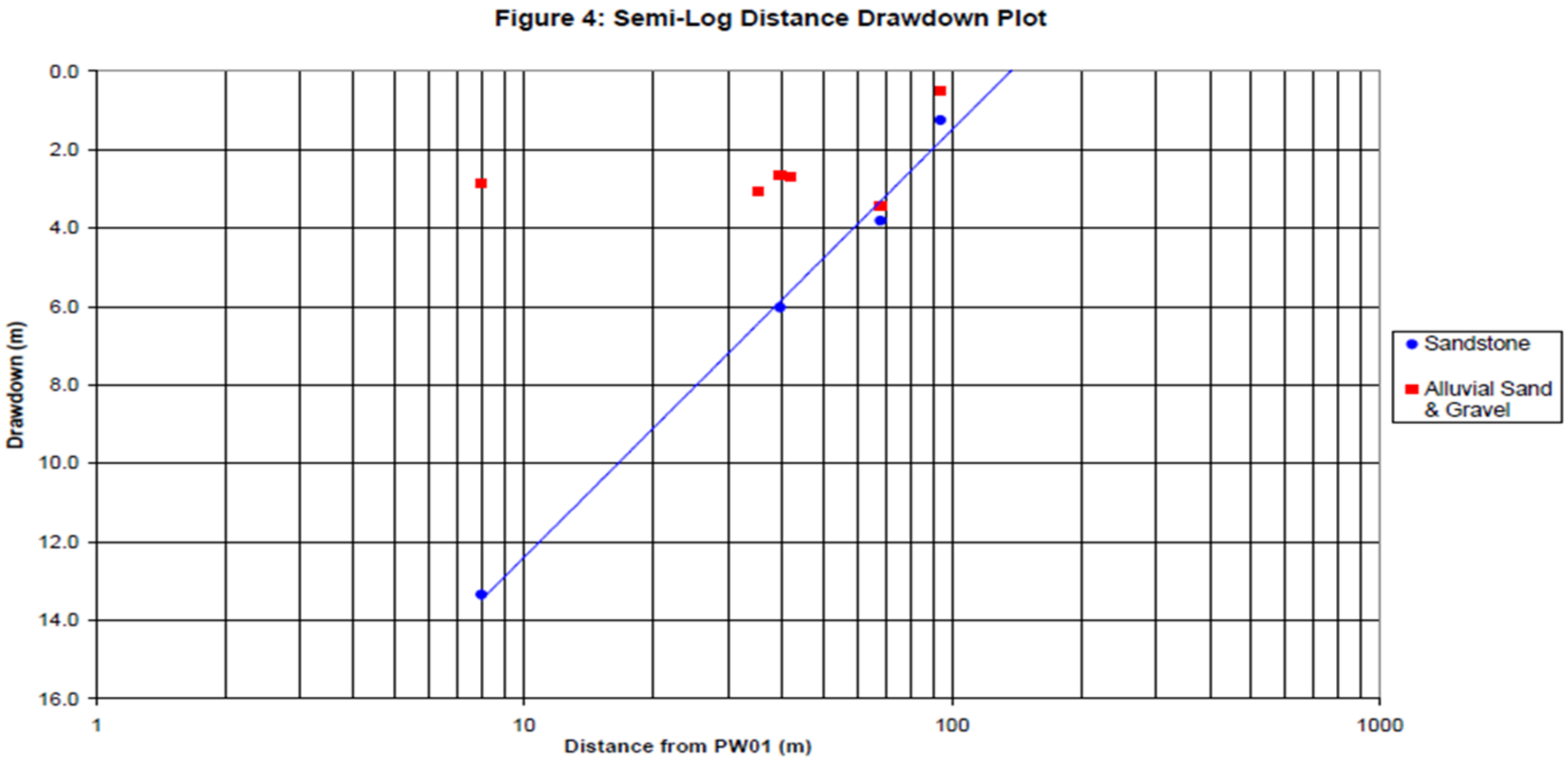
Figure 3: Semi-log Distance Drawdown Plot





WJ GROUNDWATER LIMITED	Job Title: Shaft 16 Belfast Sewers Pumping Test Layout			Drawing No. Figure 1	Rev No. 3
	Job No. J1339	Scale: 1:1000@A4	Date: 21/08/2006	Drawn By: MB	Checked by: SF

WJ GROUNDWATER LIMITED
J1339 Belfast Sewers: Shaft 16 TPS
Pumping Test



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